VOL. XXXII January, 1954 No. 1

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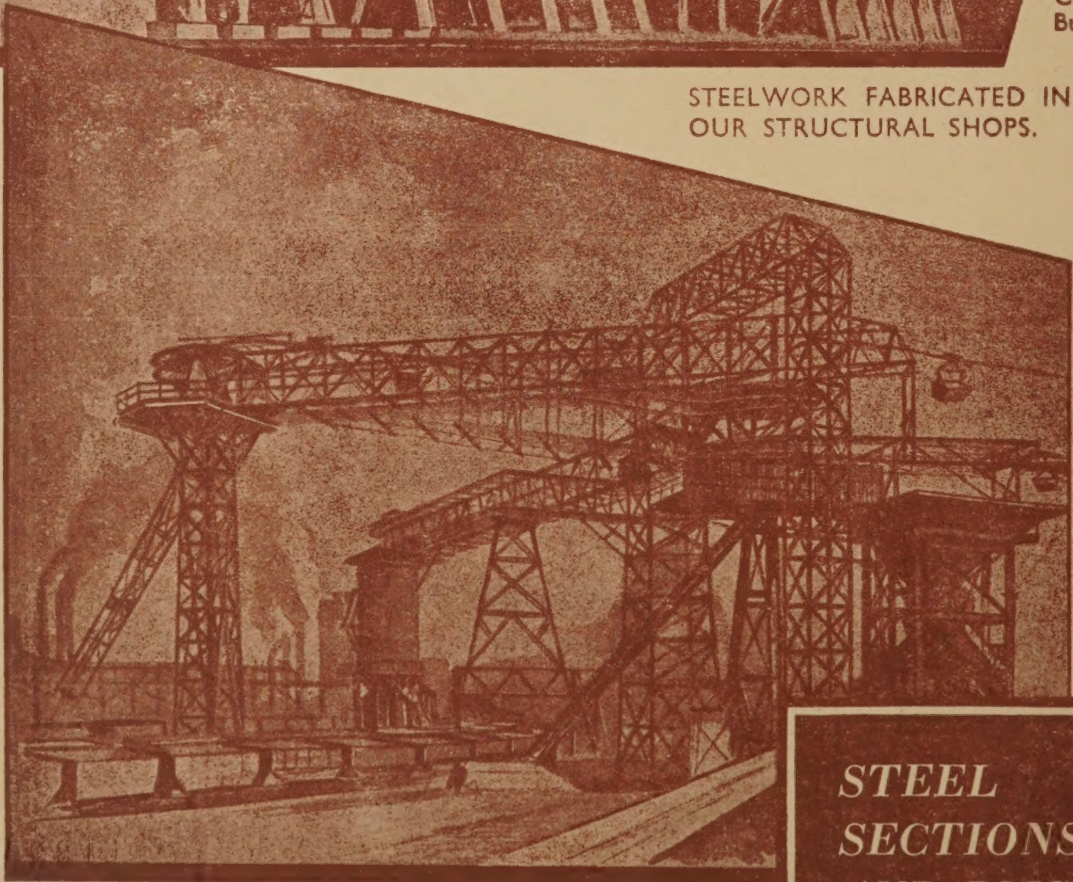

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Presidential Address*

By Lt.-Colonel R. F. Galbraith, M.C., B.Sc.(Eng.), M.I.Struct.E., A.M.I.C.E.

To-night my first task must be to thank all the nembers of the Institution for the great honour they ave conferred on me by electing me as their President or the coming Session.

I feel most deeply the importance of the task you have et me to maintain and enhance the undoubted prestige

Fig. 1

f the Institution, and I shall endeavour to justify the onfidence you have shown in me by my actions.

In addressing you this evening I propose to deal briefly vith some "Modern Developments of Structural Engineering," and my first subject will deal with oundation work.

The development in site investigation and the theory nd practice of soil mechanics during the last few years as enabled structural engineers to depart from the ather empirical rules that defined the safe bearing ressure for various types of soils and to predict with nuch greater certainty the probable sub-soil conditions nd to design accordingly.

The term "site investigation" covers all the investiations needed to decide on the suitability of one or nore sites for a specific purpose. Civil Engineering ode of Practice No. 1 sets out in detail and in a conenient form a summary of the information it is desirable o obtain, but to-night I propose to confine my remarks o the investigation of the sub-soil and its effects on design, and to exclude all consideration as to general suitability of a site for its proposed purpose.

About fifteen years ago the maximum concession made to site investigation was the excavation of a single trial hole somewhere on the proposed site and if possible where no load bearing member would be needed. To-day, site investigation commences with a study of the geology of the district and an examination of the bore-hole results recorded at South Kensington by the Geological Survey of Great Britain. This preliminary work enables the engineer to decide the number and position of trial pits or bore-holes. Trial pits tend to become expensive if carried to a depth greater than 10 to 15 feet, but in stiff ground where no timbering is required they permit a visual examination of the various strata. Shallow bore-holes can be made in light soil by a post-hole auger, whilst bore-holes of medium depth are readily made by a shell and auger boring rig (Fig. 1), or other type of rig. In the case of deep boring through rock diamond bits are used to obtain cores, driven by machines (Fig. 2) capable of drilling to a depth of 1,000 ft. Whatever type of drilling is used careful records are kept of each strata showing depth, soil type, water level, etc.

Samples are taken at various depths and are either disturbed, i.e., taken from the bore-hole and packed into airtight containers or undisturbed when care is taken to preserve the natural structure of the soil. Special sampling tools (Fig. 3) are used to obtain undisturbed samples. When boring has been completed the various engineering properties of the samples are tested in the laboratory. The shear strength of cohesive soils can be measured by a portable apparatus either in the laboratory or in the field (Fig. 4). A shear box apparatus (Fig. 5) is used to measure the shear strength of cohesive or frictional soils under various vertical pressures. A larger apparatus is available (Fig. 6) for use on specimens up to 18 in. square. The triaxial compression machine (Fig. 7) is a more elaborate machine and is used to measure shear strength of soils. The specimen in this machine is 4 in. diameter, the full size of the sample. Smaller machines are available

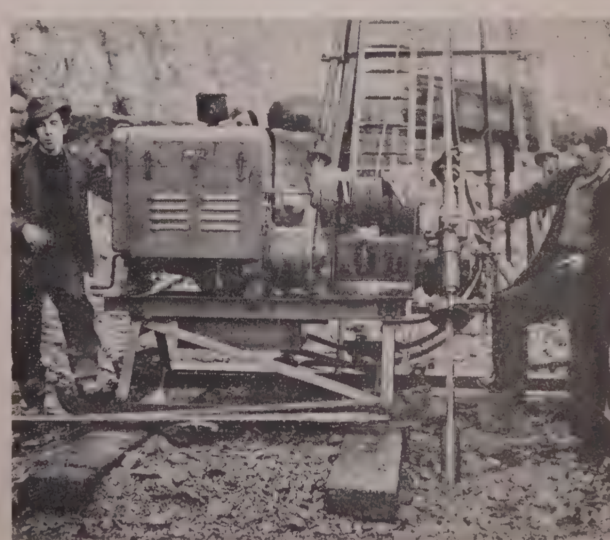


Fig. 2

*Given before a General Meeting of the Institution of Structural 'ngineers at 11, Upper Belgrave Street, London, S.W.1, on 'hursday, October 1st, 1953.

Fig. 3

Fig. 4

Fig. 5

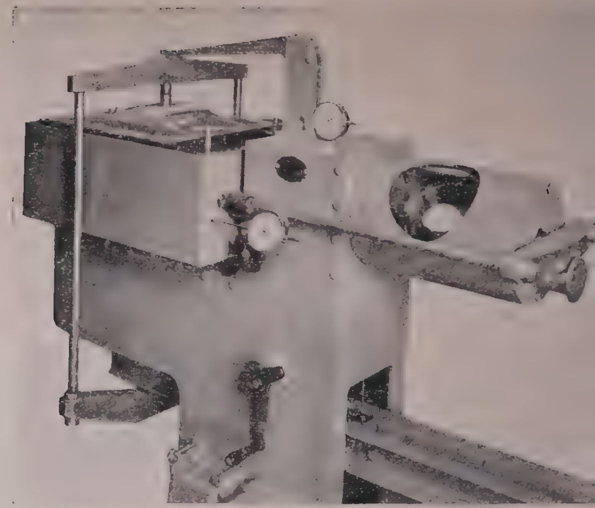


Fig. 6

using specimens cut from the 4 in. core. A typi failure (Fig. 8) is on an inclined plane, the sample hav been enclosed in a rubber membrane.

Other machines, e.g., power-driven sieves, consoli tion apparatus, etc., are used to obtain other propert of the soil. The information obtained from the bori and from laboratory tests of the mechanical propert of the soils enable the engineer to solve his foundat problems in a rational way.

In addition to borings and laboratory tests it is som times desirable or necessary to make actual bear tests on the site. For this, portable equipment moun on a lorry (Fig. 9) is used. A metal plate is forced in the ground whilst the load and penetrations are record In more important conditions jacking beams load with kentledge are arranged (Fig. 10), so that a hydrau jack can be used to force a metal plate 18 in. squ

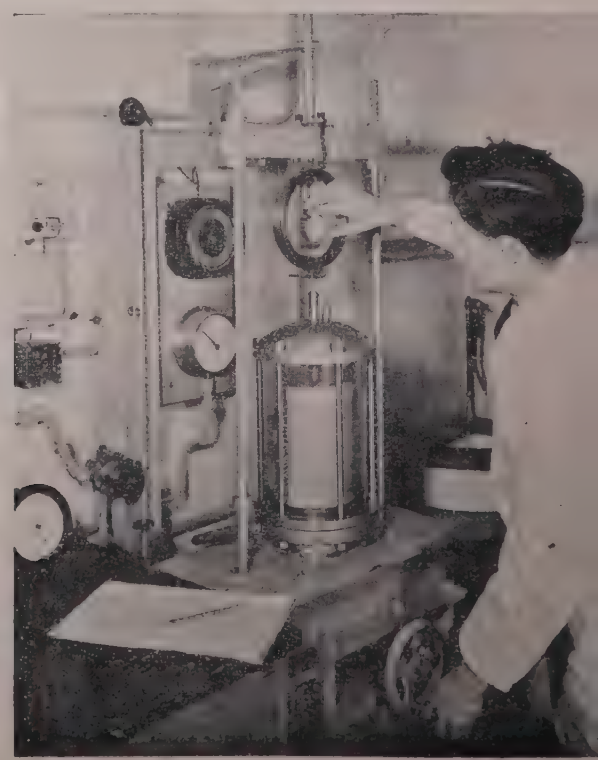


Fig. 7

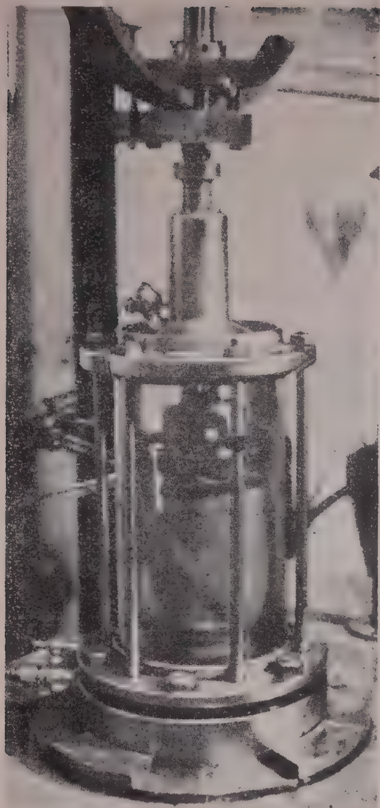


Fig. 8

Fig. 10

Fig. 11) into the ground. The settlement being measured by a precision level and scales.

The results obtained by these bearing tests need careful interpretation as they only reflect the character of the soil located within a depth of less than twice the width of the test plate, and unless the soil is uniform to a considerable depth, which is most unlikely, the results will be misleading. It is necessary to select an allowable soil pressure not only from the results of such tests, but also from the character of the soil profile and from the size of the foundation itself.

Geophysical methods to determine the depth of bedrock have been developed to assist the engineer. A geophysical survey can quickly give a general picture of the bedrock surface under the whole site enabling the subsequent drilling programme to be planned economically. In dealing with river sites where drilling is more difficult and costly than on dry land, several rock profiles can be obtained across the river by geophysical methods at the cost of a single bore-hole. The seismic refractory method and electrical resistivity method are those generally used in these cases. The results obtained by the seismic method are accurate within 10 per cent.

Considerable research continues on the theory of soil mechanics. The International Association for Soil Mechanics holds conferences periodically and publishes much information on current problems in GEOTECHNIQUE.

Turning to earth-moving machinery, the earliest record of an excavator is that of the Otis Steam Shovel (Fig. 12) designed and patented by William S. Otis and built in 1837 in Philadelphia, U.S.A. Since then there has been an enormous development in earth-moving machinery, and to-day the large walking drag line, the scraper and the trencher have made it possible to simplify excavation problems on certain sites in a remarkable way.

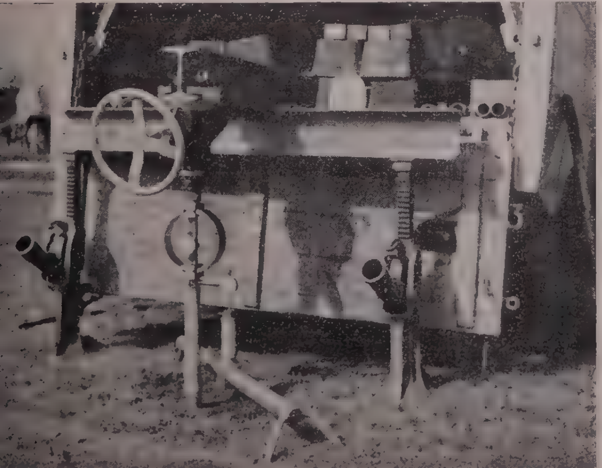


Fig. 9

Fig. 11

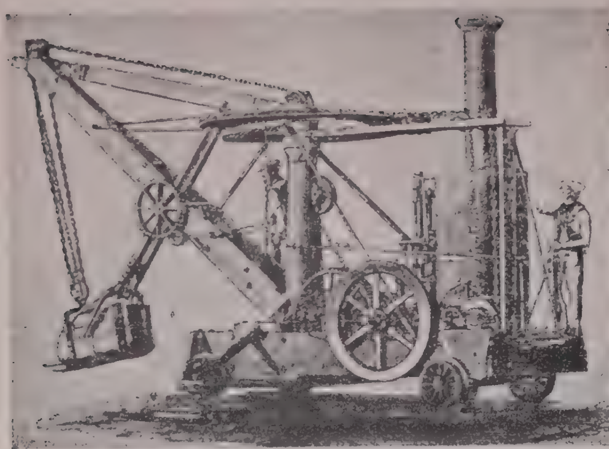


Fig. 12

In the excavation of a large basement approximately 21 ft. deep a large drag line (Fig. 13) was used at ground level. The sub-soil consisted of layers of sand and gravel with thin layers of silty brown clay overlying about 30 ft. of London blue clay with a water table about 31 ft. below the surface. No timbering was used at the side of the excavation which was battered to a slope of about 60° to the horizontal (Fig. 14). The foundations

Fig. 13

of a housing scheme can now be excavated by the vertical type of trencher (Fig. 15). This machine can deal with trenches 18 in., 21 in. and 24 in. wide and up to 8 ft. 6 in. deep. Right-angle cuts at the corners are made by the machine without hand excavation. The clearing of a site and adjustment of surface levels becomes simple when carried out by an angle dozer. These modern machines, and there are many others, save an enormous amount of hand labour in open excavation work. Hand excavation must of course be

Fig. 14

used in certain classes of work, but even here a smal mechanical crane and pneumatic spades lighten the task of the navvy excavating deep pier holes or underpinning old foundations.

The modern method of dealing with excavation in waterlogged soil is to lower the ground water level al over the site by pumping from a number of points or wells simultaneously. Filters are used to prevent sand being pumped out of gravel. Three systems are in general use—the shallow well system, where a number of bored filter wells (Fig. 16) are connected to a ring

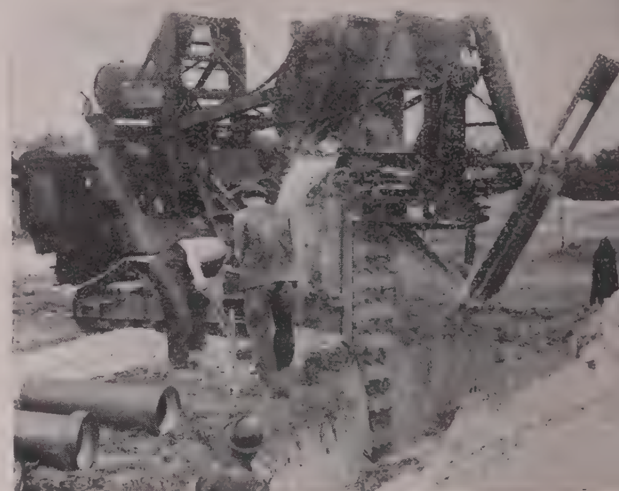


Fig. 15

suction main connected to the pumping equipment (Fig. 17) ; the well point system uses a number of smal tubes with a short filter portion jetted into position and connected by a ring main to the pumping unit (Fig. 18). With shallow wells or the well point system the maximum lowering of the water table will be about 15 ft. below the axis of the pump, but of course multi-stage pumping can be used when greater depths are required than can be reached by a single stage.

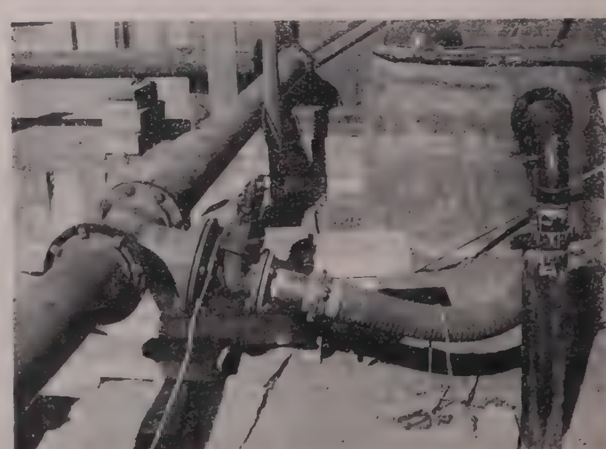


Fig. 16

The deep well system (Fig. 19) has a number of large diameter bored filter wells each equipped with its own submersible pump lowered into the filter well (Fig. 20). Deep wells are only limited by the depth of boring and the location of water-bearing strata.

Earth pressures and earth retaining structures—the wedge theory of Coulomb in 1773 and the theory of earth pressure developed by Rankine in 1857 are still

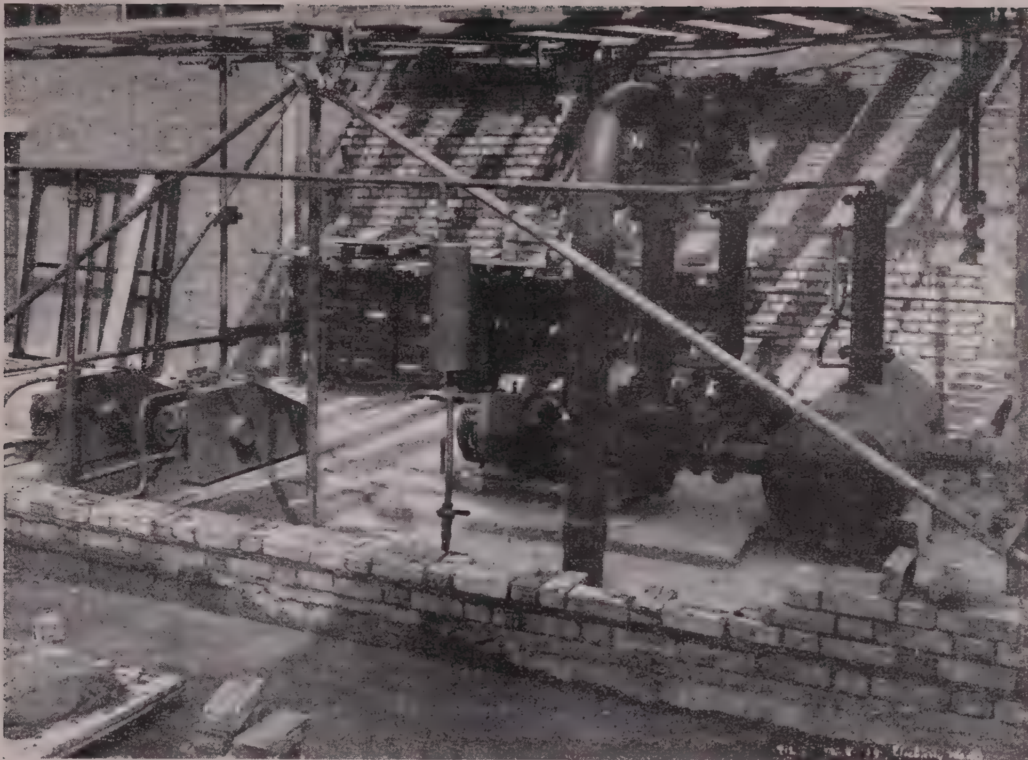


Fig. 17

egarded as applicable to non-cohesive soils such as lry sand and gravel where the angle of friction can be letermined with some accuracy. Various modifications o apply these theories to cohesive soils have been nade. Bell, in 1915, modifying the Rankine theory o deal with clays for example.

In 1951, after several years of careful study of earth ressure problems, the Civil Engineering Code of Practice Joint Committee convened by The Institution under the hairmanship of Mr. F. E. Wentworth-Sheilds issued 'Civil Engineering Code of Practice No. 2—Earth Retaining Structures." Formulæ and tables are ncluded for various types of soils which may be regarded is the latest opinion on the subject and enables engineers o calculate the probable earth pressures under certain onditions, but engineers must also reserve the right to lepart from the results obtained from any particular heory and base the design on the results of experience.

The older forms of timbering to the vertical sides of pen excavations or trenches have been somewhat modified since 1939 by the extreme shortage of timber. Steel joist walings and concrete poling boards with timber rakers have been successfully used. Interlocking

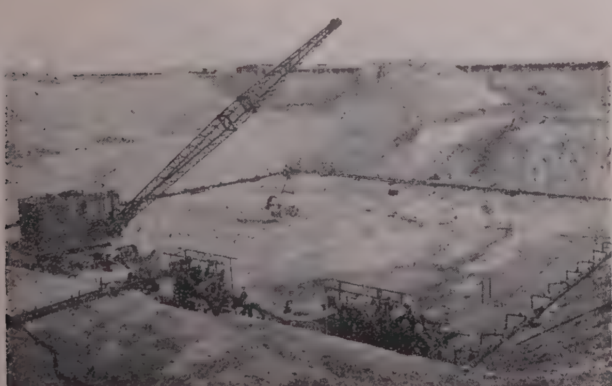


Fig. 18

Fig. 19

Fig. 20

steel sheet piling has proved invaluable in some cases of heavily waterlogged sites. The steel piling is often withdrawn after the permanent construction has been built. In many cases the use of any support has been avoided by battering the sides of the excavation to a suitable angle to avoid slip.

The methods used for permanent construction to resist earth pressure have not varied very much in the past decade or so. The reinforced concrete retaining wall is probably the most common solution to the problem in building work. In 1952, owing to the extreme shortage of steel, prestressed concrete retaining walls were used in some cases. Fig. 21 shows a retaining wall about 18 ft. high retaining stratas of sandy gravel

Fig. 22

on silt ; the wall is 18 in. thick and is post-tensioned, using the Lee McCall system with Macalloy steel bars bent into "U" form and tensioned in pairs from the ends of the "U." Fig. 22 shows the bars in position ready for formwork. The high local stresses at the base of the "U" are distributed by means of a strip of heavy gauge expanded metal (Fig. 23).

Before leaving this section I must mention the geotechnical processes which are now in use in suitable cases whereby the properties of the soil can be changed either permanently or temporarily. The most useful processes for building work are the freezing processes used for excavation in fine grain soils below ground water level, and the injection processes which includes cement grouting, clay grouting and chemical consolidation. The chemical consolidation method has proved most successful and most varied in application. The materials which are suitable for this treatment are deposits of sand or gravel, both above and below ground water level, and containing up to approximately 5 per cent. of impurities. Very fine sands, say, of effective grain size less than 0.1 millimetres are too fine for injection. Clays, silts and peats are also unsuitable, and other processes such as freezing, explosive consolidation, etc., should be used. The process of chemical consolidation relies upon the almost instantaneous reaction of two chemical solutions which binds the sand grains together into what amounts to an artificial sandstone. A silica gel is precipitated from a solution of sodium silicate by the action of a salt such as calcium chloride. The free silica acid binds the whole mass into a permanent solid which is

Fig. 23

impermeable to water. Fig. 24 shows solidified ground under a basement floor.

Structural Theory

Castigliano published his now famous treatise on the principle of least work in 1879, but little regard was paid to this until 1909 when an English translation by Mr. Ewart S. Andrews, one of our Past-Presidents, was published. Professor Hardy Cross published his moment distribution method in 1929. Since then an ever-increasing stream of theories, methods and analogies have been put forward by various authors, many of whom indulge in a considerable amount of mathematical gymnastics to avoid the fundamental fact that the calculations for a redundant structure, apart from the very simplest cases, must of reality be a complicated business. However, a careful analysis, backed by sound engineering judgment and experience, may well repay the engineer by producing an economic design.

It is unfortunate that the work of the "Steel Structures Research Committee," who published a final

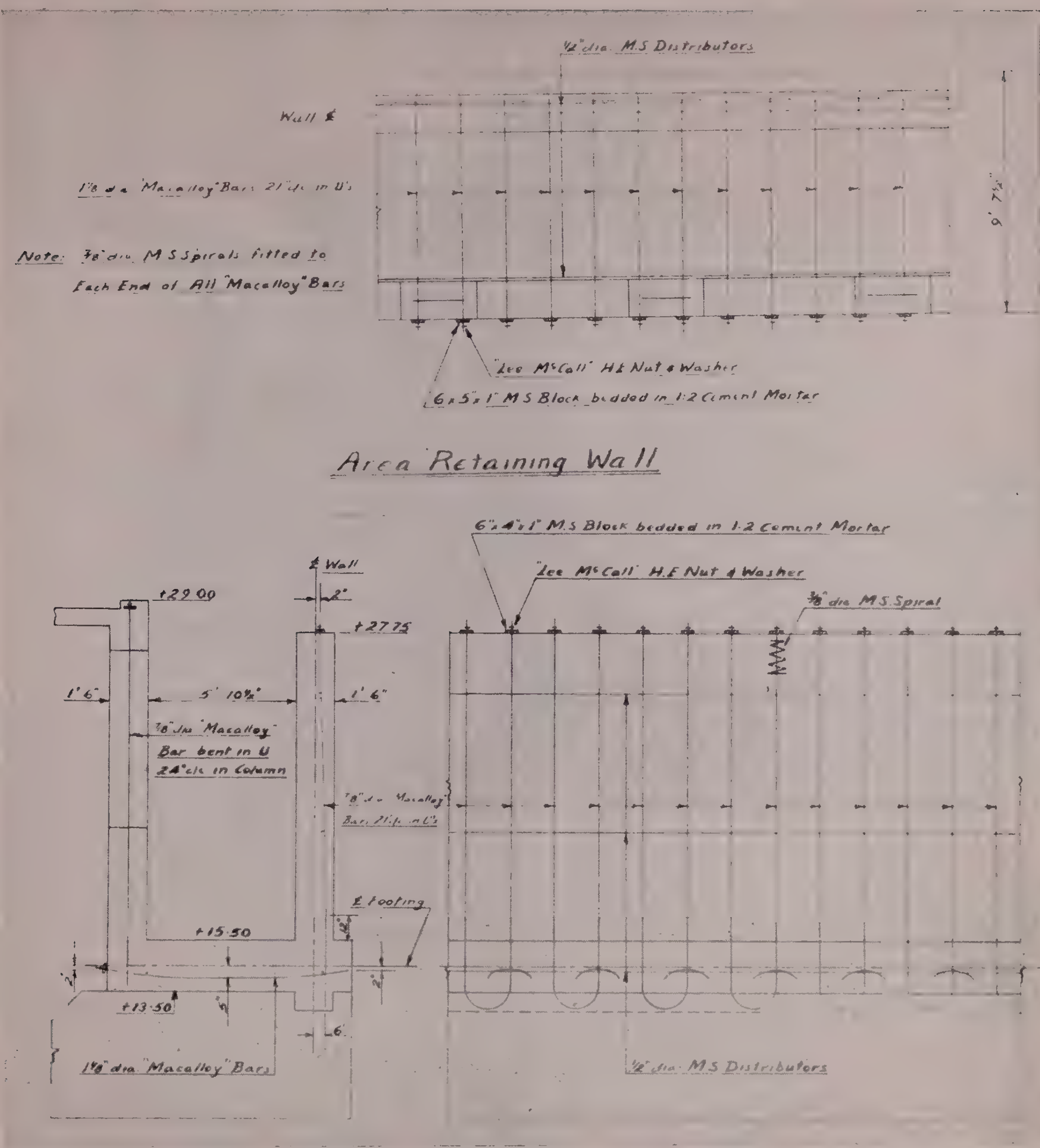


Fig. 21

eport in 1937, has not been integrated into engineering ractice, as the rational methods put forward in the eport would undoubtedly lead to economy. Each mplifying step made to meet criticisms of complexity as meant a loss of economy.

When the Committee stopped work, experimental ork was started in the Civil Engineering Department f the University of Bristol, and later at Cambridge. n 1944 Professor J. F. Baker and his team of experts t the Engineering Laboratory of the University of ambridge developed what is now known as the "Plastic heory." This is an outstanding contribution to modern hought, and the award of the Institution Gold Medal to rofessor Baker in 1952 for his work on the Steel Strucures Research Committee and in connection with the Plastic Theory," pays a fitting tribute to this.

The results obtained by this theory are illustrated in striking manner in the case of the 60 ft. standard torage shed designed for the War Department by rofessor Baker and his team at Cambridge in 1951. he original War Department design (Fig. 25) uses

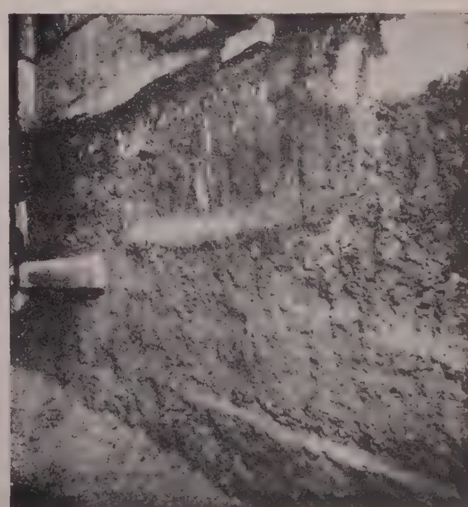


Fig. 24

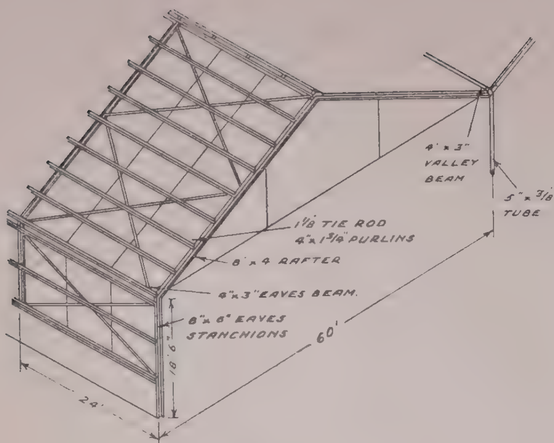


Fig. 25

4.54 pounds of steel per square foot of floor area, whilst the Cambridge design (Fig. 26) requires only 3.1 pounds per square foot, i.e., a saving of 34 per cent. The table of comparative weights and sizes (Fig. 27) is worthy of study by all engineers.

Professor A. L. L. Baker has put forward a plastic theory of design for ordinary reinforced concrete and prestressed concrete and for shell roofs. Considerable experimental work in support of this theory has been carried out, and further work is proceeding at Imperial College, University of London.

There has been an increasing use of plastic models to study the deformations of statically indeterminate structures such as portal frames. The equipment used can be very simple—a drawing-board, pins, a model of the structure being examined and a scale are sufficient, but more elaborate equipment such as Professor Beggs' deformeter can be used. In either case the results obtained by the use of such methods are comparable to the results obtained by more rigorous analysis. Fig. 28 shows the outlines of a two-hinged portal bridge, 48 ft. span carrying Ministry of Transport loading, and the table of calculations to obtain the value of the horizontal reaction. In Fig. 29 the outline of the model used is shown, together with the deflections and experimental readings. A comparison of the influence lines obtained by calculation and experiment show the maximum difference of just below 4 per cent.

The Codes of Practice which have been issued by the Council of Codes of Practice for Buildings and the Civil Engineering Codes of Practice Joint Committee on various subjects represent a standard of good practice in normal cases. Many of our members have served on the Committees drafting these Codes and among the most important of those already published are CP 3 Chapter V Loading, CP 113 Structural Use of Steel in Buildings, and CP 114 Structural Use of Normal Reinforced Concrete in Buildings. Also Civil Engineering Code of Practice No. 2 Earth Retaining Structures.

The fact that British Standards Specification No. 449 and the new L.C.C. Building Byelaws follow very closely to these Codes is a measure of their success, but I must appeal to the engineers not to become "Code Bound." These Codes cover normal conditions, but there are occasions when it is desirable and may even be essential to depart from the letter of the Codes. In such cases, let the engineer be bold and make the necessary departure, using his knowledge and experience to guide him to a correct solution, and if possible proving his

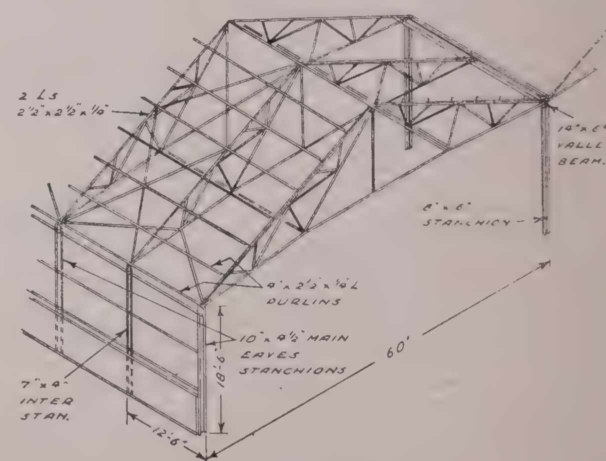


Fig. 26

TABLE 1
(60 ft. Spans)

Convential Truss and Column design (420 × 212.5 ft.)			Plastic Design (420 × 208 ft.)		
Component	Section	Wt. (tons)	Component	Section	Wt. (tons)
Trusses		68.99	Rafters	8 × 4 in. at 18 lb. R.S.J.	32.88
Longitudinal Ties	3 × 2 × ¼ in. L at 4.04 lb.	5.37	Horizontal Ties with Sag rods	1⅛ in. dia. at 338 lb. ⅜ in. dia. at 0.376 lb.	6.04
Purlins	4 × 2½ × ¼ in. L at 5.32 lb. 3½ × 2½ × ¼ in. L at 4.89 lb.	14.13 38.97	Purlins	4 × 1¾ in. at 5 lb. R.S.J. 4¾ × 1¾ in. at 6.5 lb. R.S.J.	39.00 16.90
Sheeting Rails	3 × 2½ × ¼ in. L at 4.46 lb.	2.54	Sheeting Rails	4 × 1¾ in, at 5 lb. R.S.J.	1.86
Eaves Stanchions	10 × 4½ in. at 25 lb. R.S.J.	7.43	Eaves Stanchions	8 × 6 in. at 35 lb. R.S.J.	5.29
Internal Stanchions	5 × 6 in. at 35 lb. R.S.J.	17.34	Internal Stanchions	5 × ⅜ in. at 18 lb. Tubes	8.25
Valley Beams	14 × 6 in. at 46 lb. R.S.J.	26.18	Eaves and Valley Beams	4 × 3 in. at 10 lb. R.S.J.	7.43
Total weight : 181.0 tons			Total weight : 117.6 tons		
Weight per sq. ft. of Plan : 4.543 lb.			Weight per sq. ft. of Plan : 3.015 lb.		

Fig. 27

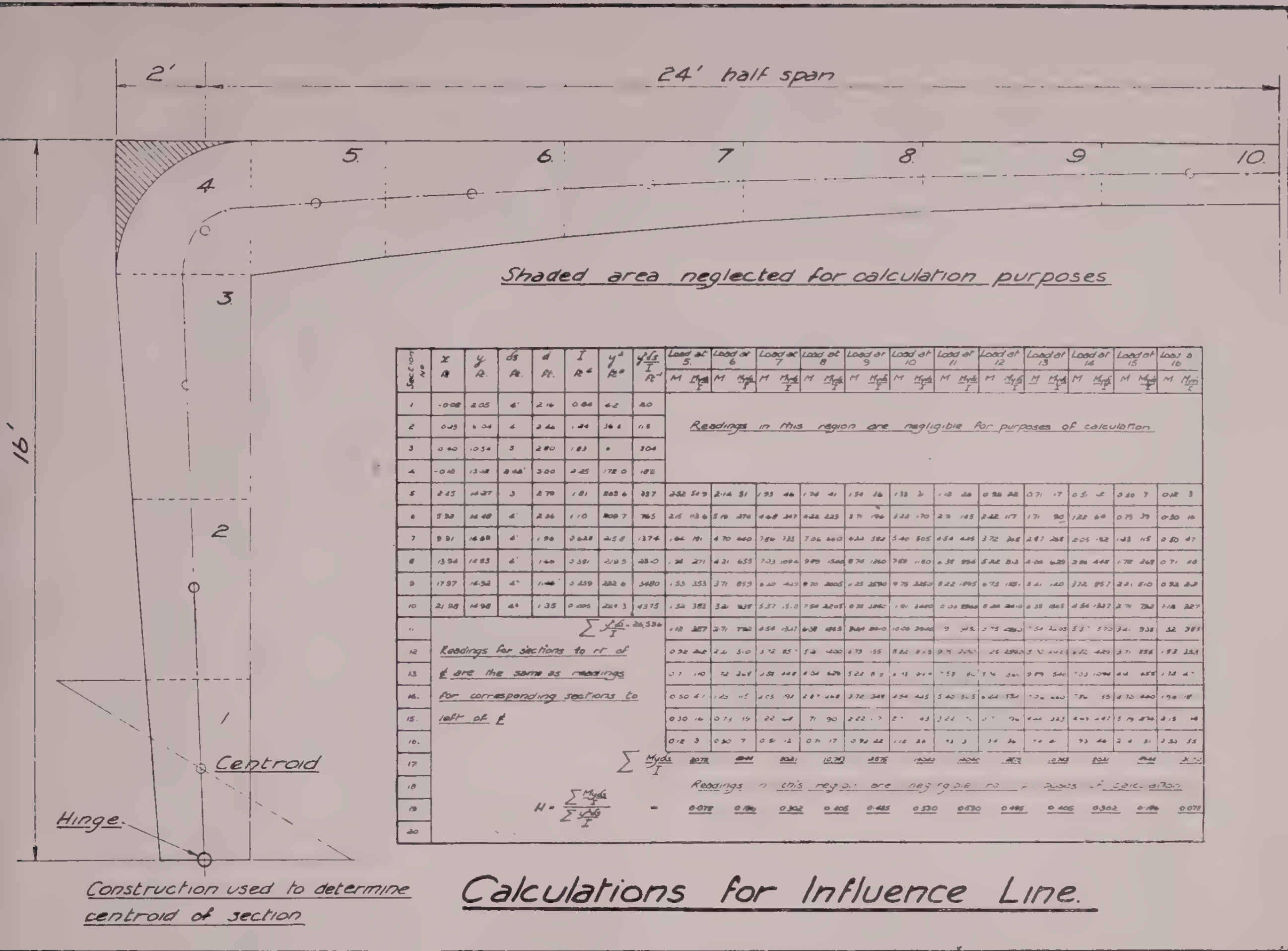
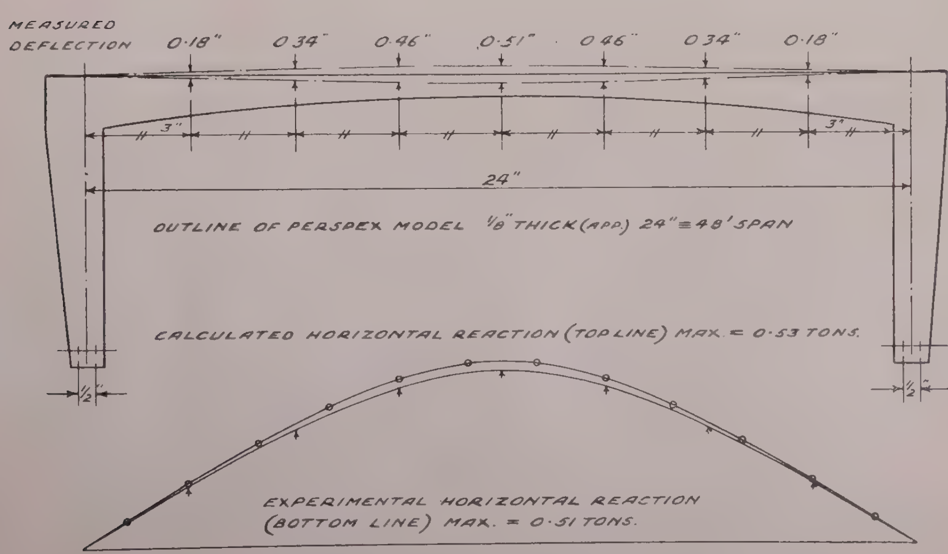


Fig. 28



INFLUENCE LINE FOR HORIZONTAL REACTION H.

Fig. 29

Fig. 30 [Crown Coypright Reserved

actions by large-scale tests, as only by such a course can he hope to achieve progress.

Structural Steelwork

In the field of riveted or bolted work there is little to report. The new B.S. sections give an improved range of sizes, workshop practice has achieved a high degree of efficiency, and the output of structural shops has only been restricted by the shortage of steel and in some cases by the lack of orders.

High Tensile steel to B.S.S. 548 has been used with economy in a number of buildings carrying heavy floor loads. The initial extra cost of approximately 10 per cent. of the structural steelwork is offset by a saving of perhaps 30 per cent. in the weight due to the increased working stresses permissible. The use of a high tensile rolled section may avoid the use of a compound section with a consequent saving in cost. In addition, high tensile steel will generally result in increased headroom and smaller concrete casings.

Fig. 31

The case of welded steelwork has been one of rapid development. Since the first report on welded steel structures was issued by the Institution in 1935 there has been continual research by several important organisations, and considerable improvement both in the technique of welding and in the manufacture of electrodes. Perhaps one of the most outstanding applications of welding has been the Bailey Bridge designed by Mr. D. Bailey, now Sir Donald, one of our members. The units are entirely of welded construction and during the war were manufactured in many works both large and small. The negligible proportions o misfits when in service overseas pays a high tribut both to the firms and workmen engaged in the manufacture, and to the inspecting officers of the Ministry of Supply.

The Freeman Bridge over the Rhine (Fig. 30) and th Festival of Britain Bridge over the Thames (Fig. 31 are but two examples of the many uses Bailey Bridgin has been put to both in war and in peace.

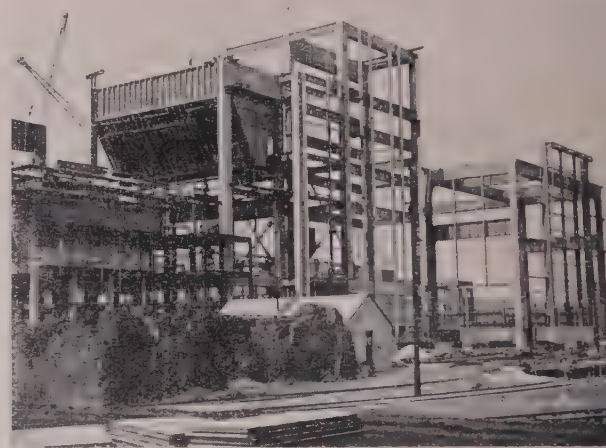


Fig. 34

Welding lends itself to clean shapes and profiles whicl may clearly be seen among the following typical examples of structural steelwork :—

1. Warehouse for the Port of Bristol Authority (Fig. 32)—a riveted and bolted design.
2. Stella South Power Station on the Tyne (Fig. 33)— an all-welded steel structure with box type main columns and roof girders. The five coal bunkers (Fig. 34) eacl have a capacity of 1,250 tons, while the five boilers are suspended from the main building frame. The building is 440 ft. long and 266 ft. wide between the main columns with a height of 145 ft. to the roof level.

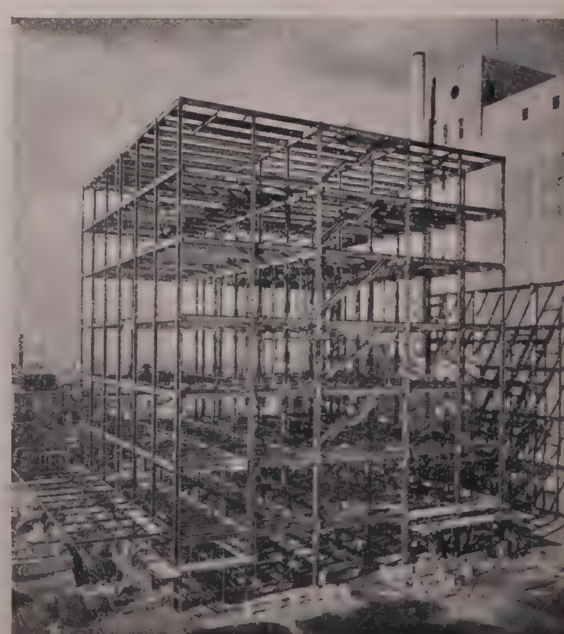


Fig. 35

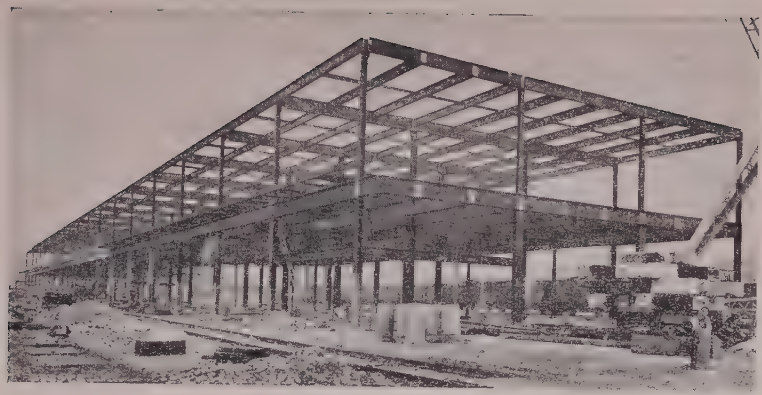


Fig. 32

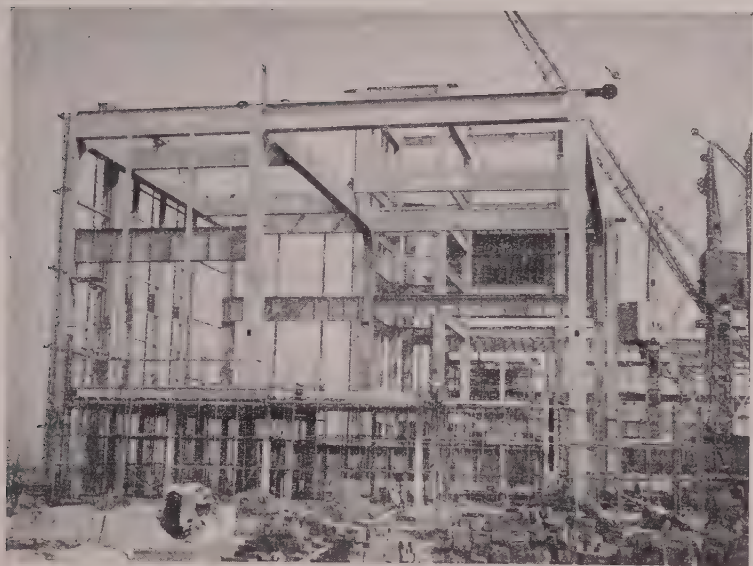


Fig. 33

Fig. 36

Fig. 37

Fig. 38

3. Building for Messrs. Spillers at Hull (Fig. 35)—designed in high tensile steel to B.S.S. 548 with high tensile rivets.

4. Modern welded Vierendeel girder to a weaving shed (Fig. 36).

5. New tube factory at Liverpool (Fig. 37)—an all-welded single-storey factory, comprising two bays 90 ft. wide and 1,530 ft. long, with a clear height to the eaves of 30 ft. The total weight of steel is 4,000 tons. The main rafters have a profile tapering from 26 in. deep at the bottom of the slope to 14 in. deep at the top. These were made from 20 in. × 12 in. broad flange beams. The webs were cut diagonally, the pieces reversed and the webs butt welded together. The stanchion bases (Fig. 38) were welded up and bolted to the foundation blocks. All site joints in the porta frames (Fig. 39) were welded. The general site joint were made with black bolts and no special provisior was made for expansion, except in the crane girders These joints were placed at one-third points along th length of the building and were designed with a ga corresponding to the linear expansion over a range o temperature of 60°F. The movement was dealt witl by machined bearing plates sliding on phosphor bronz strips. The crane rail had a special 10 per cent. beve cut across the tread to permit a very small actual gap

6. Factory at Wood Green (Fig. 40)—heavy fille joist construction carrying a safe load of 4 cwt./sq. ft All compounds and main girders are in high tensil steel. The stanchions in the outer wall shown on th left do not support the full load from incoming girder as they are encased in reinforced concrete.

7. Welded construction—(a) Welded column (Fig. 41 during fabrication in welding shop for the I.C.I. plan at Wilton. (b) Welded portal with side girder at tw levels (Fig. 42) for I.C.I. at Wilton. (c) Twin rivetec girders supported on brackets of welded column a British Nylon Spinners works at Pontypool (Fig. 43).

Fig. 39

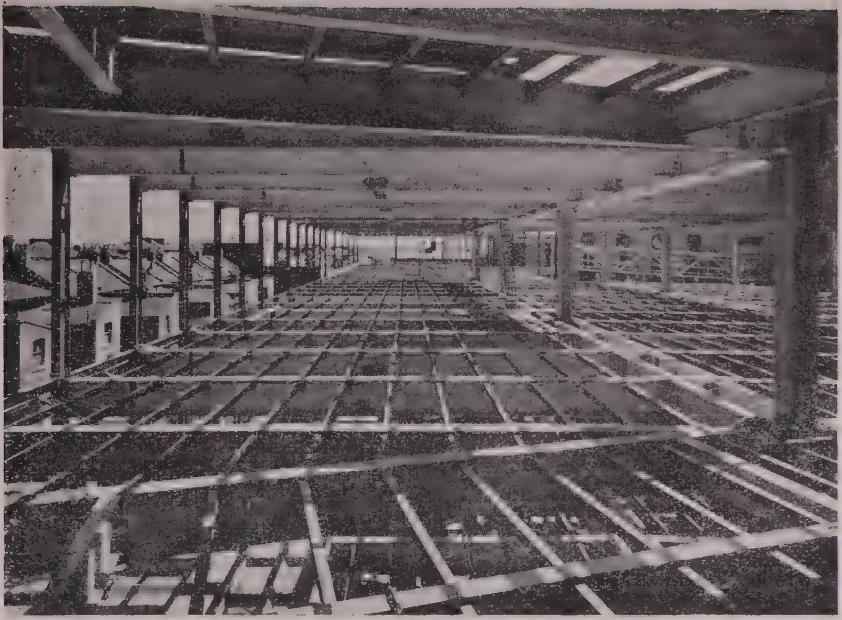


Fig. 40

8. Power station at Shotton (Fig. 44)—the columns nd crane girders are riveted with welded roof frames, ll connections being riveted at the site.

9. 180 ft. clear span hangar at Bristol (Fig. 45) and a elicopter test rig. This is the only one of its kind in he country, and can be seen through the open door of he hangar. A close-up view is shown at Fig. 46.

10. All-welded steel frame building at Barnsley. The gs used for shop welding the main frames (Fig. 47) lustrate modern technique. The detail of the knee ints (Fig. 48) is of considerable interest.

Before leaving structural steelwork I should like to how a very recent development—prestressed steel. Ve all know of prestressed concrete, but there has been ery little work done on prestressed steel. If the rinciples are correct for concrete there seems no reason why they should not achieve an economy in steel when applied to structural steelwork. The first published use of prestressed steel was by Professor Magnel in 1950. Mr. R. A. Sefton Jenkins has now designed a 60 ft. span lattice girder (Fig. 49) for the Harlow Development Corporation which has a prestressed bottom core. Full details of this work will be described by Mr. Sefton Jenkins later in the Session when he reads a Paper on this subject.

Reinforced Concrete

The progress of reinforced concrete during the last decade has been one of steadily increasing volume due to severe steel shortage with little change in basic design. The recently published report on economy of building materials recommends the use of a higher

Fig. 41

Fig. 42

working stress, i.e., 20,000 lb./sq. in. for m.s. reinforcing bars, but doubts appear in the minds of some engineers as to the justification for this increase, apart from the grounds of economy.

The use of deformed bars, either cold twisted bars or twin bars, has been advocated as a means of saving steel by using higher stresses, and a report on the use of deformed steel bars in reinforced concrete has recently been published by the Institution. This certainly does reduce the total quantity of steel required, but the fact that the increased stress will result in an increased strain, with a consequent increase in the total width of cracking, must be borne in mind. Special steps must be taken in all structures exposed to the weather to design so that a large number of fine cracks will develop rather than a few large cracks. This can often be obtained by intro ducing secondary reinforcement near to the surface.

Fig. 43

The cracking of concrete may result in very seriou damage to the steel reinforcement, and in addition t the need to minimise the amount of cracking in concret by skilful design there also remains the need to protec the steel reinforcement by adequate cover. In protecte situations, or where the concrete has an addition protection from a stone or brick facing, and where ther is a low fire risk it may be possible to reduce the cove recommended by the Code of Practice, but in expose outside situations I feel it is necessary to increase th cover ; even 2 in. may not be an unduly large amoun

The introduction of power-driven bending machine and the improved training given to steel reinforcemen workers has ensured that the engineer's requirement for steel bars bent to special shapes can readily be me Welding has been used to eliminate hooks and anchor ages, and to fabricate the reinforcement of certai reinforced concrete members. This is particularl suitable for precast units. Fig. 50 shows the reinforce ment for a precast portal frame welded up ready to b placed in the mould.

The skill and ingenuity of the formworker has bee called into play by the extreme shortage of timber sinc 1939. Steel forms, metal props and resin bonded ply wood sheets have all been utilised in the constructio of formwork. The cost of formwork often represents third of the total cost of reinforced concrete work, an the designer can often help to reduce this proportion b careful consideration of the shapes and sizes of hi members.

Considerable attention has been paid in recent years t the actual mix used to obtain concrete of a given quality It seems desirable that we should specify the minimur strength required at 3, 7 and 28 days, together with an special water cement ratio required, and then leave th contractor to adjust the proportions used according t the aggregate, cement and plant available. This woul tend to decrease the cost of concrete by not using con crete of a higher strength than is actually required, an not obtaining a higher grade than needed by specifyin definite proportions. A properly made 1 : 2 : 4 nomina mix to-day may well have a 28-day strength of 5,000 lb sq. in., whereas the Code requirement is 3,000 lb./sq. in and the working strength 760 or 1,000 lb./sq. in.

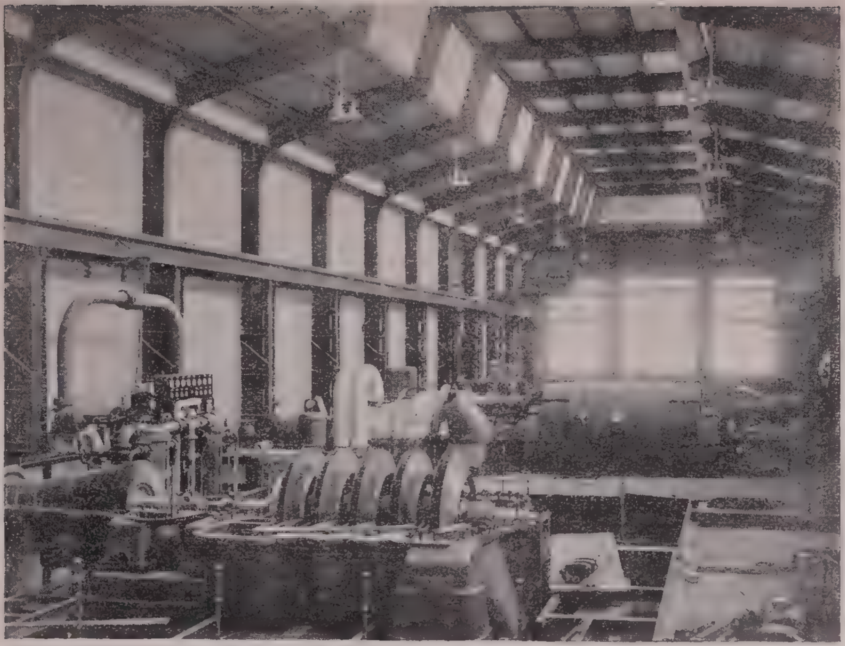


Fig. 44

Consideration should also be given to the age of con-
rete when its full working load will be applied. The
oncrete, say, in a lower length of column may well be
ine to twelve months old before it carries its full design
ead loading, and even much older before its full live
ad will be applied. Yet the size of the column is
roportioned on the basis of the 28 days strength of the
oncrete. It seems desirable to permit the use of
igher working stresses for those parts of a structure
hich by virtue of their position must be highly matured
efore they can receive their full design load, care being
aken of course to avoid accidental loading.

Mixing plant also plays a large part in the quality of
oncrete, and there has been considerable improvement
the design of such plant in recent years. Fig. 51
ows a small weigh batching plant in conjunction with a 14/10 mixer. This enables materials to be weighed and proportioned, and coupled with strict water control enables the engineer's specification to be met with accuracy.

Fig. 52 shows a small independent weigh batcher in use with two portable mixers, whilst for a very large site a central mixing plant (Fig. 53) with mechanical storage hoppers fed by belt conveyors, central cement compartment, weigh batcher and twin tilting mixers can produce high-grade concrete at a remarkable rate.

The handling of wet concrete on a site has also received attention. Small power-driven barrows (Fig. 54) and the new type of mobile crane (Fig. 55) and (Fig. 56) are of great use to the contractor. The concrete pump also has its field of use. It is possible to pump concrete at the rate of 8 to 10 yds. per hour, 125 ft. vertically, or

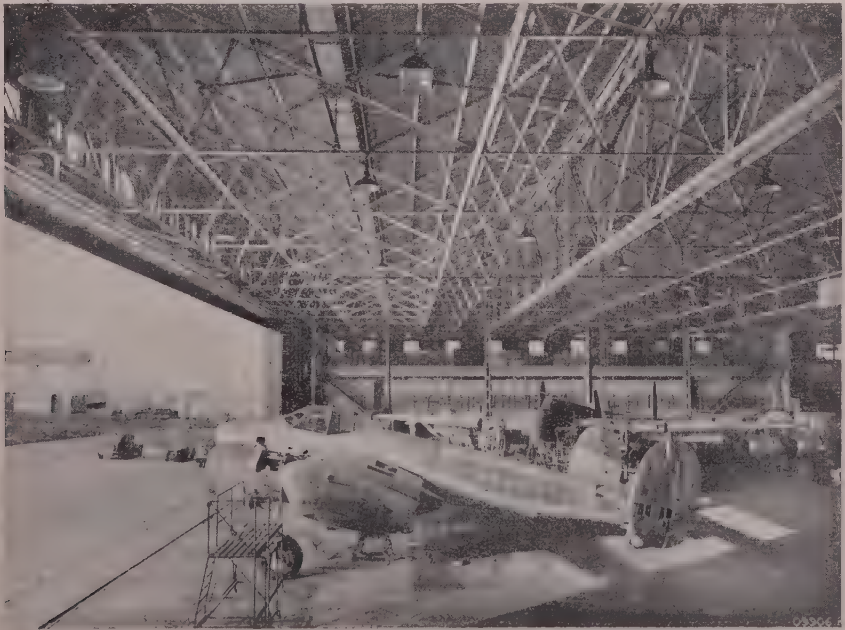


Fig. 45

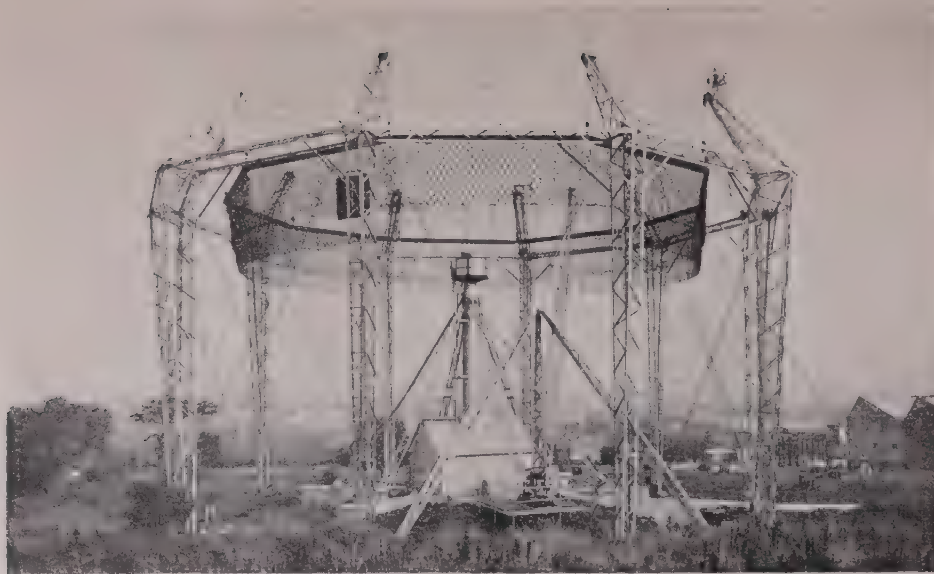


Fig. 46

Fig. 47

1,500 ft. horizontally and to deliver the concrete by pipe-line just where it is required. Concrete which can be pumped must of necessity be good concrete.

The following designs carried out in reinforced concrete illustrates the recent trends in the use of this material :—

I. A large culvert (Fig. 57) at Hemel Hempstead with a concrete sewer alongside.

II. Victoria College, Cairo (Fig. 58), designed by a British architect, is a reinforced concrete structure of very modern design. The classroom block shown is of *in situ* framing, the " brises-soleil " are constructed with vertical members of precast concrete and horizontal members cast *in situ*.

III. Fig. 59 shows a two-way slab with restrained corners designed in accordance with Code of Practice 114, and (Fig. 60) a vierendeel beam at Plashet School at East Ham.

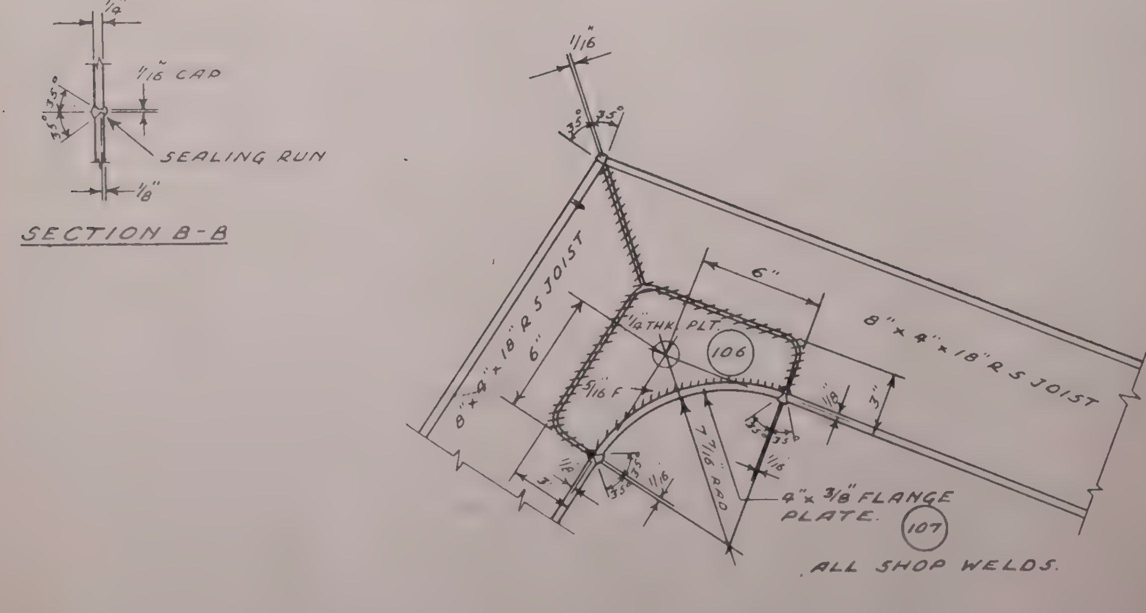


Fig. 48

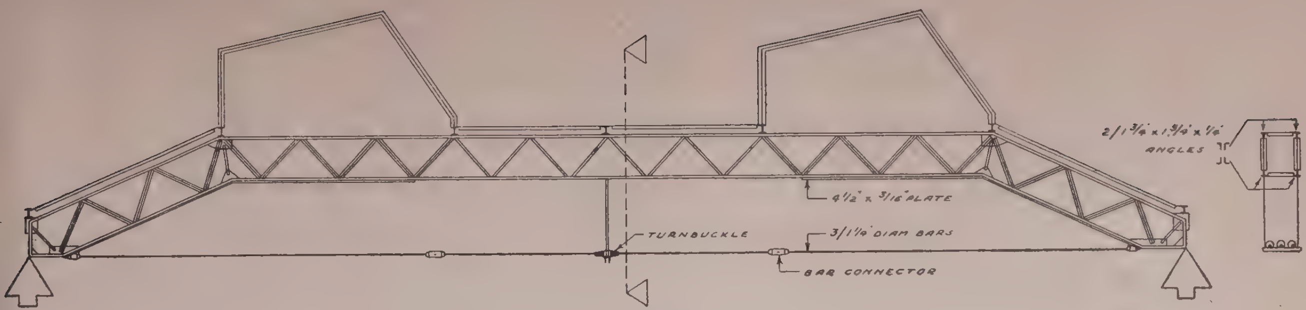


Fig. 49

IV. Technical College, Peterborough (Fig. 61) shows circular column with circular mushroom head and flat roof construction.

V. Abel House, Westminster (Fig. 62), a large office block in the course of construction.

VI. A large store at Hull (Fig. 63) is in normal reinforced concrete construction with wide span and deep T beams.

VII. Kent and Sussex Hospital (Fig. 64) circular ramp fire escapes.

Prestressed Concrete

Although early attempts had been made to improve reinforced concrete by applying tension to the steel reinforcement it was not until the early 1900's that Monsieur Eugene Freyssinet, the celebrated French engineer, studied prestressing and evolved a theory.

It was another twenty years before he could put his theory into practice. He had realised the importance of creep and in 1928 Faber and Glanville published the results of their research carried out in England on the creep of concrete, so confirming Freysinnet's deductions, and enabling him to formulate and perfect his theory of prestressing. The use of prestressing as a practical form of construction really dates from this time. Since then many developments have occurred, but all fall into two main divisions, pretensioning and post-tensioning. Pretensioning implies that the steel reinforcement is first tensioned to the required stress, concrete is then cast around the reinforcement, and becomes bonded to it and finally, when the concrete is sufficiently matured, the steel is released from its anchorage and the stress in the steel transferred to the concrete. The usual method of pretensioning is known as the " long line method," in

Fig. 50

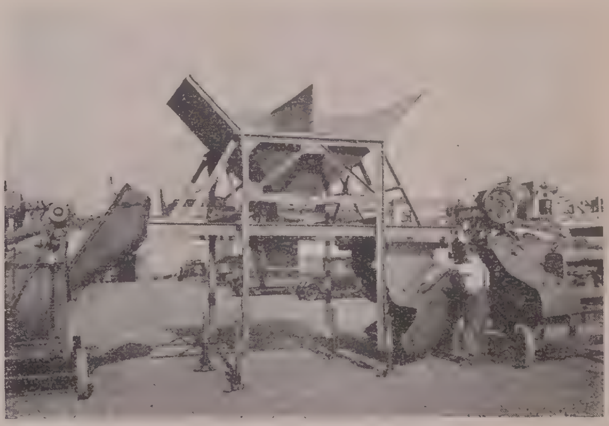


Fig. 52

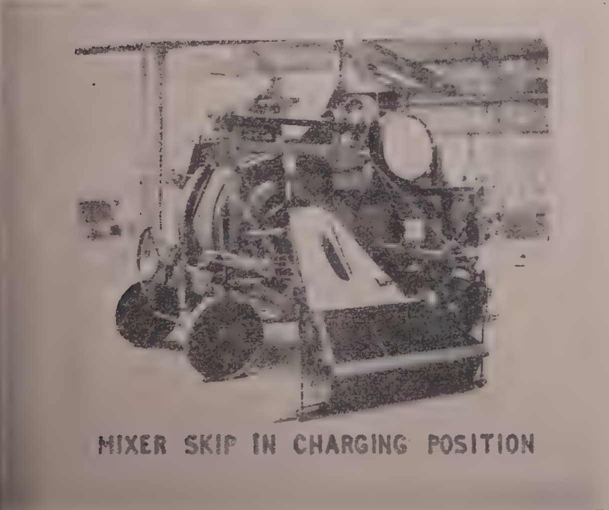


Fig. 51

Fig. 53

Fig. 54

which the steel wires are stretched between anchorages often 300 ft. apart. The concrete units are then cast around these wires with a small space between each. After hardening the wires are cut between each unit and the stress transferred. In making small units sometimes the steel wires are anchored at one end of the mould and tensioned from the other. This tends to speed up production and, in the event of a slip occurring at the anchorage only one unit would be affected.

Post-tensioning on the other hand implies that the concrete is first cast and matured, the steel wires are either placed in the mould in a tube before concreting or else threaded through a hole left in the concrete, and then tensioned and anchored at the ends of the concrete unit. Much ingenuity has been devoted to the means

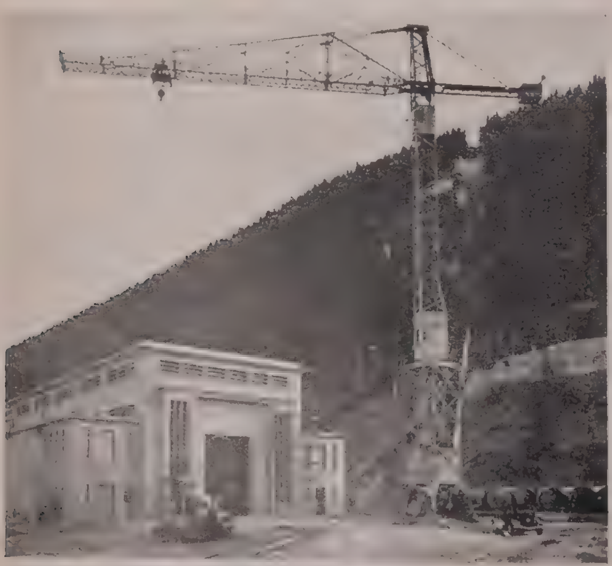


Fig. 55

of applying tension to the wires and the methods of anchoring them. Two main systems being those devised by Monsieur Freyssinet and Professor Magnel.

The third system of post-tensioning is the Lee McCall system using high tensile alloy bars instead of wires, with special threaded ends and nuts, which develop the full strength of the bar used. Whichever system is used high-grade concrete and steel are required. The production of high-grade concrete means careful control, both in mixing, placing and consolidation. Vibration is nearly always used and often some kind of artificial curing, such as steam, is used to speed up production and release the forms or moulds.

The steel wire usually used is either 8 gauge or .2 in diameter, drawn steel wire with a breaking stress of 110 to 120 tons per sq. in. It is usually supplied in large coils 8 ft. diameter to avoid the difficulties of straightening.

Some representative examples of prestressed structures are :—

(i) Castle Bridge, Shrewsbury (Fig. 65).

(ii) A prestressed column (Fig. 66)—12 in. square with one $\frac{3}{4}$ in. diameter Macalloy bar carrying a load of 60 tons at Plashet School.

Fig. 56

(iii) Post Office Manager's Office, Willesden (Fig. 67)—the prestressed concrete beams used in conjunction with heavy reinforced concrete columns can be clearly seen.

(iv) Prestressed concrete bridge at Hunter's Inn, North Devon (Fig. 68)—the precast sections are assem

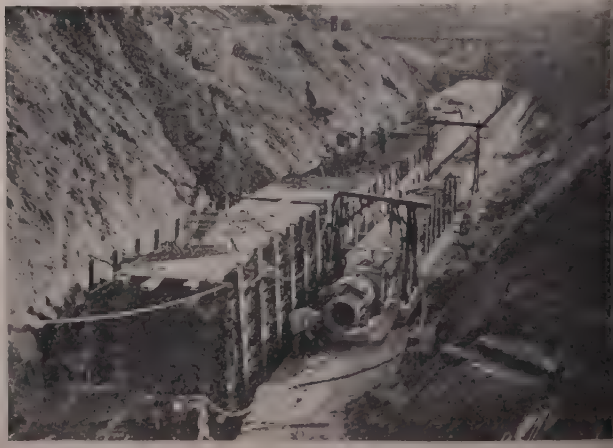


Fig. 57

Fig. 58

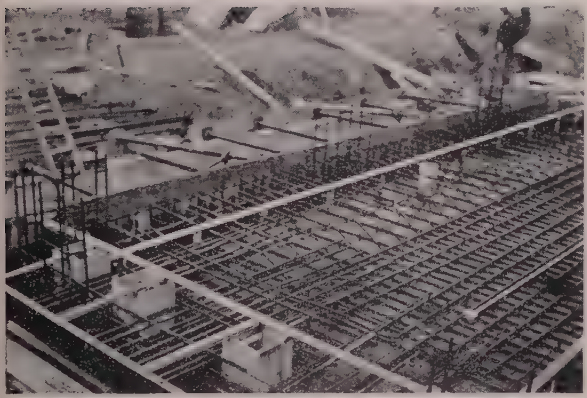


Fig. 59

bled on temporary supports before prestressing and are afterwards filled with concrete.

(v) West Green Baptist Church, Crawley (Fig. 69)—a framed prestressed concrete church.

Precast Concrete Construction

The precasting either at a factory or near the site of units in reinforced concrete or prestressed concrete has become a favoured method of construction, particularly when there are a considerable number of similar units needed.

Factory-made units have several advantages over those cast on the site. The concrete is usually made under very close control from aggregates of known characteristics. The placing and consolidation processes are on well-tried lines, while the constructions of the moulds is carried out by men well versed in this work with proper wood-working and machine shops to support them. Thus the factory-made unit will normally be of very high grade concrete, within the specified tolerances of size, and properly finished and matured.

Fig. 60

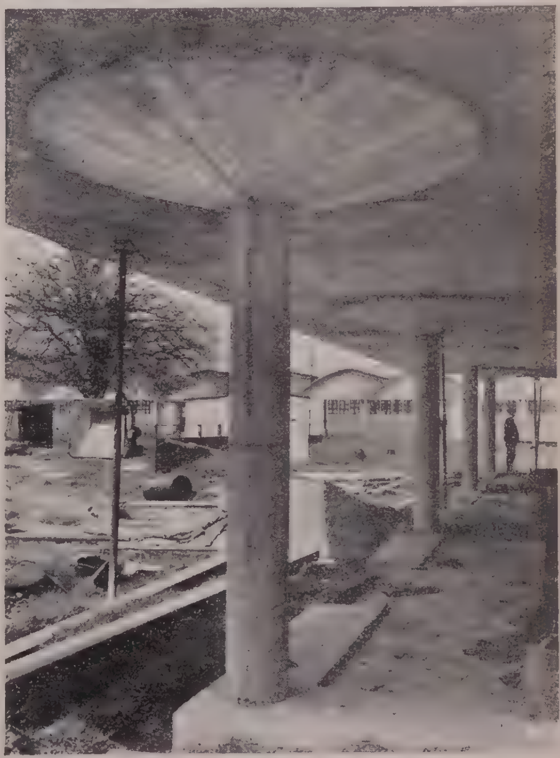


Fig. 61

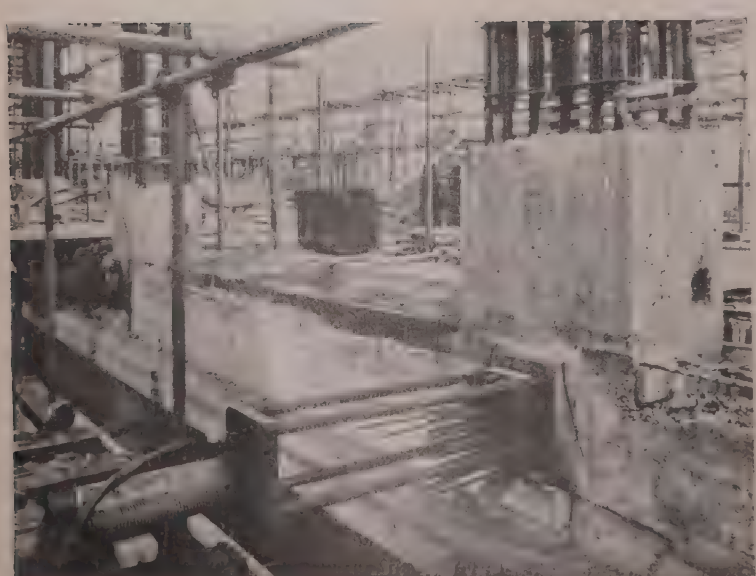
On the other hand there is, of course, the disadvantage of the cost of transport to the site.

During the war many hundreds of Ministry of Works huts were used by the Forces for camps and stores. The concrete members were cast in convenient sections and bolted together in position. Single-storey factory frames, both single and multiple spans (Fig. 70) have been developed by various manufacturers, parts being held in stock to give speedy delivery.

During the last three or four years multiple-storey buildings have been erected with precast members,

(Left)
Fig. 62

(Right)
Fig. 63



(Left)
Fig. 67

Fig. 64

Hatfield Technical College (Fig. 71) is of two storeys generally.

Plashet County Secondary School for Girls, East Ham, (Fig. 72), has six storeys of precast members, all similar. The portal frames at each end of each wing consist of a pair of precast concrete posts (Fig. 73) with projecting haunches, with mild steel reinforcement, connected to a precast prestressed beam. The joints in the vertical posts occur approximately at mid-storey height and are halved and bolted, while the joint between the projecting haunch and the central beam is a mechanical hinge joint (Fig. 74). Each portal frame is connected to the adjacent concrete wall and between the two sets of portals, precast prestressed window frames (Fig. 75) are fixed forming the walls of the classrooms. The floors which are hollow prestressed units, span between the portal frames. The stairs (Fig. 76) in the centre of the building are all precast prestressed units of approximately 20 ft. span. This system of construction allowed very rapid erection on the site, a complete floor including the ancillary *in situ* work was erected in seven working days. Fig. 77 shows a stack of 47 ft. span roof beams for the gymnasium and assembly halls. These are I-section cast in one piece and reinforced with one 1 in. diameter Macalloy bar. All the prestressed units at this school used the Lee McCall system, with Macalloy bars, mild steel anchor blocks and high efficiency nuts.

The size of precast members is really only limited by the capacity of the hoisting equipment available at the works, and at the site, and by the means of transport. The heaviest unit at Plashet School was the large window frame weighing $6\frac{1}{2}$ tons. Very heavy members weighing up to 30 tons have been successfully cast, close to their final position, and hoisted by heavy lifting tackle.

Fig. 65

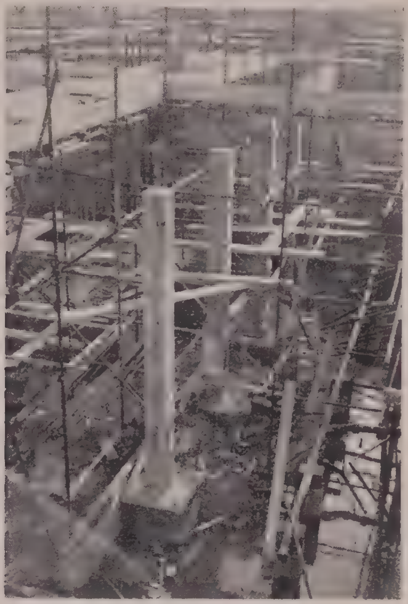


Fig. 66

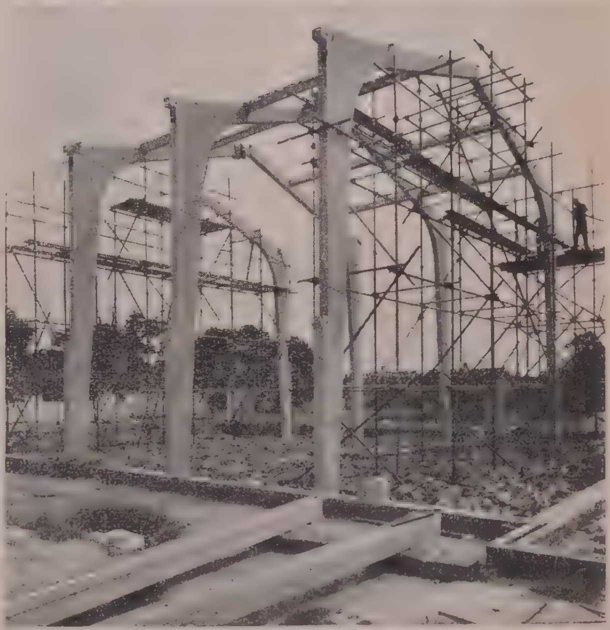


Fig. 69

Fig. 78 shows a heavy girder at Acton Power Station being lifted into position.

An unusual combination of precast and *in situ* construction is shown in Fig. 79. A new hangar near Rome, 366 ft. long and 120 ft. wide.

Before leaving precast construction I should mention the precast pile, which is now made in large quantities

Fig. 68

to a given standard and held in stock by various manufacturers, so that immediate delivery can be given for urgent jobs of well-matured piles. Many precast prestressed piles have also been used recently with success. A 12 in. × 12 in. square prestressed pile can be produced and handled up to 55 ft. long.

Shell Roofs

The use of shell roofs originated in Germany. The general theory was evolved by U. Finsterwalder and F. Dischinger, whilst Professor A. L. L. Baker, of Imperial College, has proposed a simple method of designing shell roofs based on the ultimate strength theory which has been confirmed by tests. The following are a few examples of shell roof construction :—

A. Malden Manor Station, Surrey (Fig. 80)—an early example of shell roof construction.

B. A bus garage at Peckham (Fig. 81)—three-way reinforcement and trimming bars for the rectangular roof lights can be clearly seen.

C. Sewage Works at Colne Valley (Fig. 82)—a good example of modern shell roof construction. The interior (Fig. 83) is particularly pleasing.

D. Cereal Factory at Dagenham (Fig. 84).

E. Royal Marine Barracks, Deal (Fig. 85).

Tubular Construction

The structural use of steel tubes is another development which has been of particular advantage during the steel shortage. Roof trusses, space frames, portal buildings have all been carried out in this form of construction. In May, 1953, a Code of Practice (C.P. 113 : 201 (1953)) was issued giving recommendations for the design and fabrication and erection of steel

Fig. 70

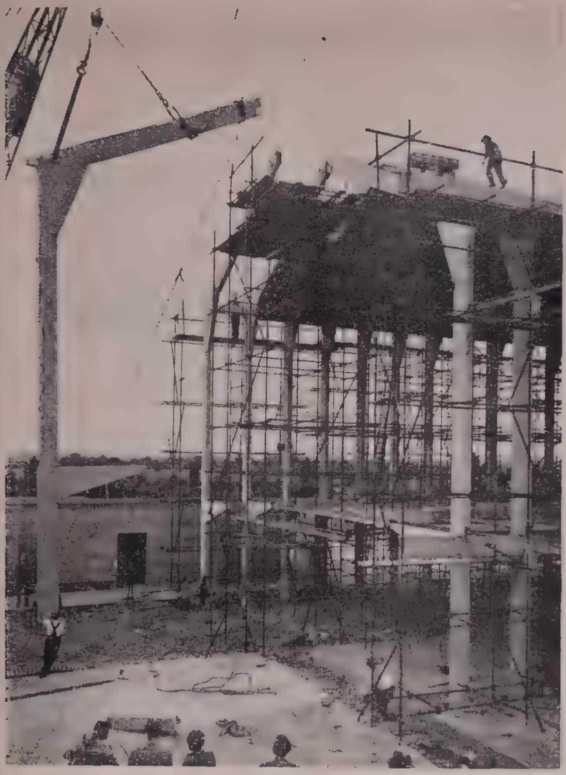


Fig. 71

Fig. 73

Fig. 74

Fig. 75

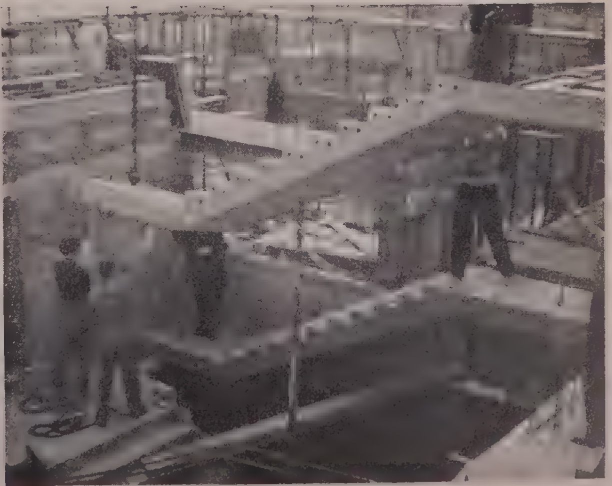


Fig. 76

Fig. 77

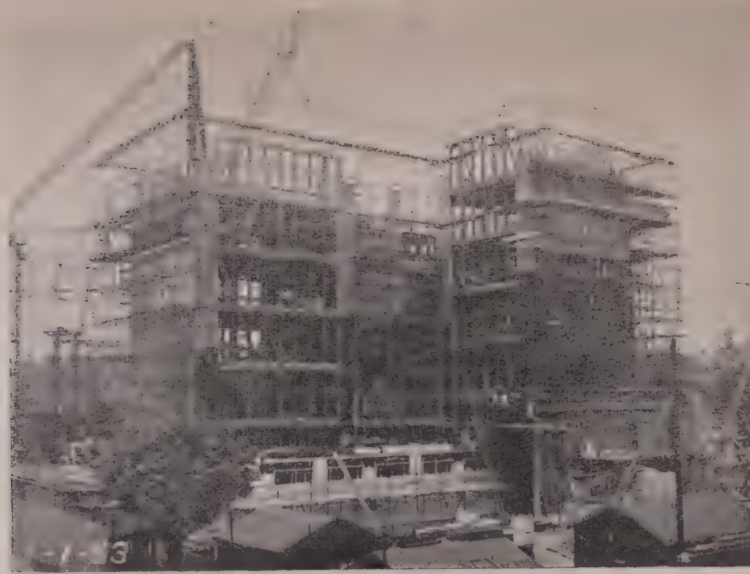


Fig. 72

tubes. During recent months we have seen an immense amount of temporary construction carried out in steel tubes for work in connection with the Coronation of our Queen, but perhaps the most spectacular use of tubes is the W.1400 drag-line (Fig. 86) installed by Messrs. Stewarts & Lloyds near Corby, Northamptonshire. This is the biggest machine of its kind in the world. It weighs 1,600 tons and in its working position the head of the jib is 175 ft. above the ground, i.e., higher than Nelson's Column in Trafalgar Square. The great jib (Fig. 87) is 282 ft. long and is constructed of high tensile steel tubes in triangulated form. The steel is a special one evolved by the Reseaıch and Technical Development Department of Stewarts and Lloyds. It is a high tensile chromium-molybdenum fine grain steel, aluminium killed. It is designed to giv better tensile properties, a higher yield point, to hav less tendency to harden and to be suitable for welding with a McQaid Ehn grain size of grade 6 or finer. Th machine was designed and constructed under th responsibility of Ransome & Rapier, Ltd., whilst the ji was designed by Tubewright Ltd., and is a remarkabl achievement in tubular construction, unique both i its conception and design, and is an outstanding exampl of British skill and enterprise.

Aluminium

Aluminium, or rather, the aluminium alloys, hav become a very definite structural material. The alloy bulk for bulk are about one-third the weight of steel an

Fig. 78

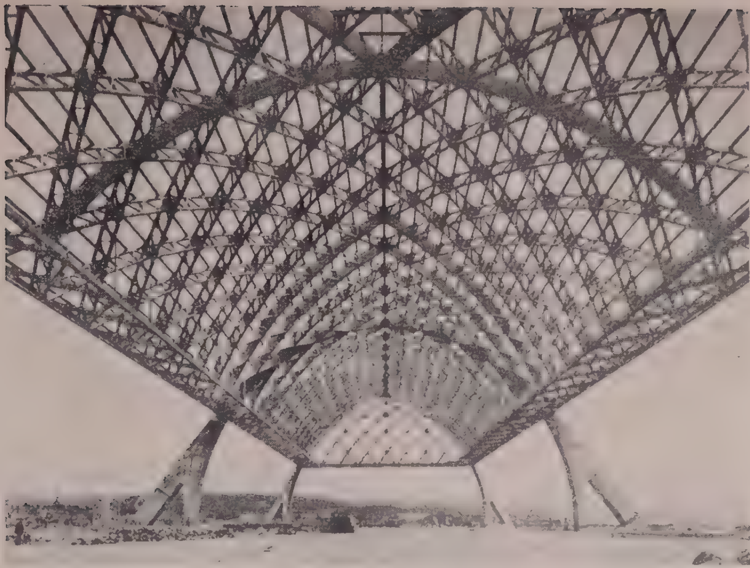


Fig. 79

re from a half to three-quarters the strength of steel. n any structure where the dead weight of construction orms a high proportion of the total load aluminium lloy becomes a competitive material. Small units uch as roof trusses for housing, where repetition allows he use of mass production methods, form another field here the light alloys can hold their own.

Apart from the cost aspect no maintenance is required. ight, graceful members can be constructed, and no urface decoration is needed. The Institution's report n the "Structural Use of Aluminium Alloys in Buildngs," published in 1950, gives much information on he design and use of aluminium.

The following representative examples show the possibilities in the use of this material :—

(a) A large storage building (Fig. 88) for export to the Persian Gulf. The trial erection is shown of part of the 300 ft. long building, three 50 ft. bays wide, carrying 5-ton overhead cranes. The material is A.W.10 throughout.

(b) 35 tons mono-tower crane (Fig. 89) for Alex. Stevenson & Sons' shipyard at Glasgow. The jib (Fig. 90) is 130 ft. long. The longitudinal members are in H.E.15-WP. The cross bracing in H.E.10-WP, the rivets are $\frac{3}{4}$ in. diameter and $\frac{7}{8}$ in. diameter recessed in N.6 and H.E.10-WP.

Fig. 80

Fig. 83

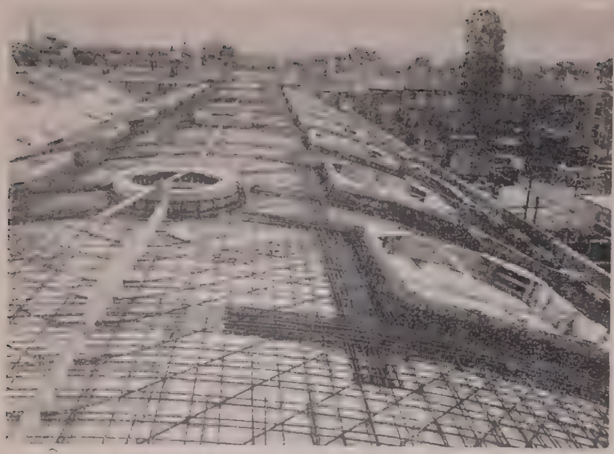


Fig. 81

Fig. 82

Fig. 84

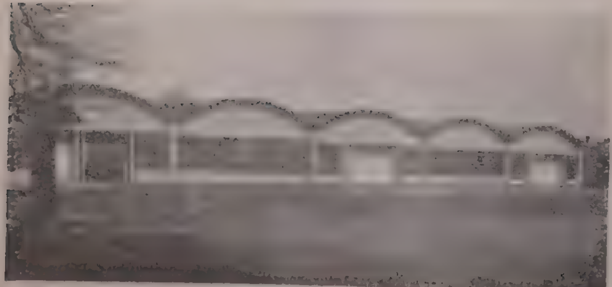


Fig. 85

Fig. 86

Fig. 87

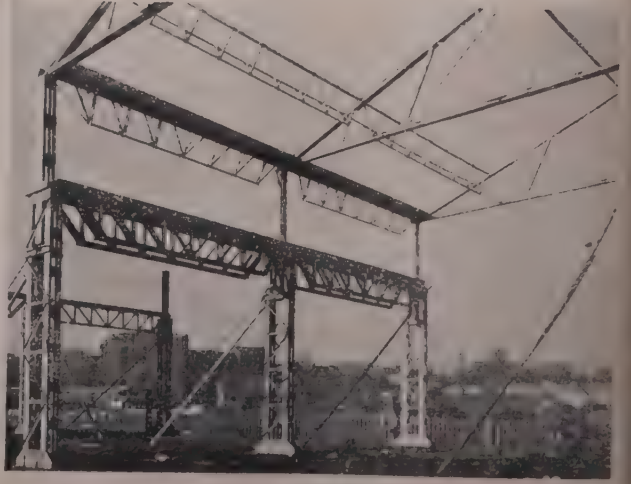


Fig. 88

Fig. 89

c) Space frames for roof (Fig. 91) for the Aero search Ltd., Duxford. These frames use standard l special extrusion. They are 42 ft. long and can be ed by two men.

d) Plate girders for bus garage (Fig. 92). These are ft. span and approximately 6 ft. deep. The webs and

Fig. 90

Fig. 91

flange plates are in noral 65 SWP alloy and the angles are in noral 57 SWP extrusions.

(e) Hangar for the Comet at De Havillands Ltd. (Fig. 93). This graceful structure is of 200 ft. clear span. The main framing was erected in thirteen weeks by eighteen men using two 5-ton hand-operated cranes.

(f) Board Mills, South Wales (Fig. 94). An aluminium roof construction is supported on prestressed concrete

Fig. 92

columns and shows a useful combination of two modern developments.

(g) Aberdeen Harbour Movable Bridge (Fig. 95). The new double-leaf heel trunnion bascule bridge spanning the 70 ft. entrance to the Victoria Dock, which was opened yesterday by Her Majesty Queen Elizabeth the Queen Mother.

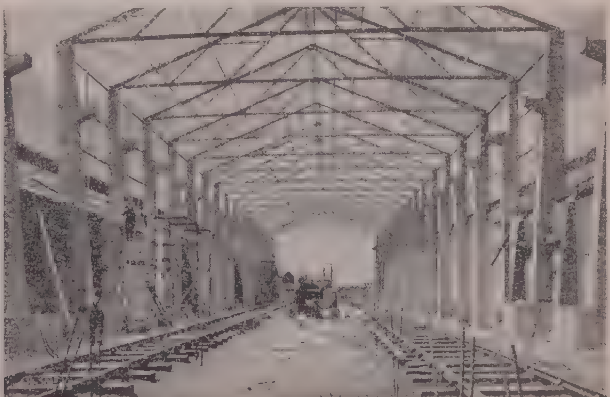


Fig. 94

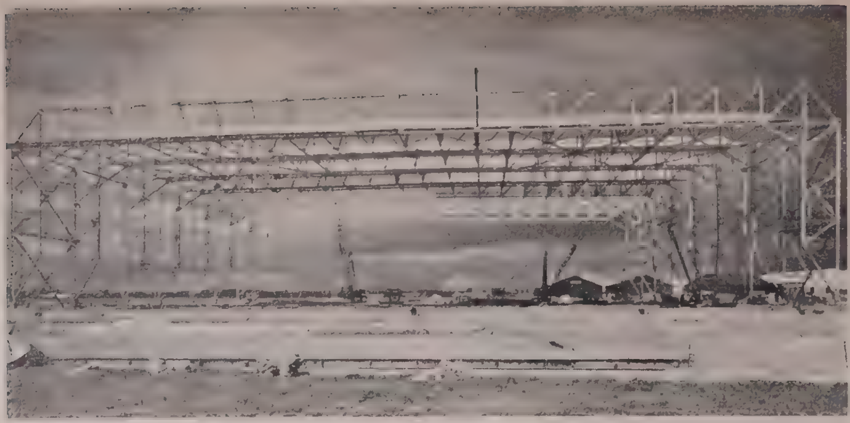


Fig. 93

Novel features are involved in the balancing and operation of the bridge and in the fabrication of the moving leaves. This unorthodox design is the subject of a patent by Mr. John Anderson, the Harbour Engineer.

The bridge has a 22 ft. wide roadway between the trusses, carrying two lanes of road traffic. Provision is also made for single line rail traffic, whilst outside the trusses 5 ft. wide walkways are provided to cope with the peak hour pedestrian traffic.

Each moving leaf weighs approximately 45 tons, such a low weight being achieved mainly due to the use of aluminium alloys. It rotates about a trunnion at its landward end, the trunnions being set at 100 ft. centres. The angle of opening is 87° and the approximate time of opening and closing is $1\frac{1}{4}$ minutes. Electrical control gear is fitted, whilst in the event of an electrical power failure, hand gear may be used to operate the bridge.

The shore steel fixed structure performs the functions of carrying the counterbalance weight pulley, transmitting a portion of the live load from the bridge well away from the quay face and embodying the operating machinery pin type rack quadrant.

The mass concrete foundations occupy the minimum ground space and there are pits to accommodate the counterbalance weights and stop arms of the truss, when the bridge is in the fully open position.

The foregoing details give a brief picture of the salient features of the movable bridge, which is the first of its type in the world.

Future Developments

It is hard to forecast the direction that technical developments will take in the future, but there are several needs in the structural engineering world whic no doubt will be met in the future. Concrete which ha a high crushing strength is a heavy material. Th light-weight concretes produced by air entrainment a not really strong enough for many structural purpose so there remains a demand for a light-weight concret of high strength. If it can be produced at a competitiv cost the scope for its use in structural engineering immense. The great defect of concrete, mainly its hig dead weight, would disappear and precast membe could be increased in size without a comparative increas in the capacity of lifting tackle. Perhaps the solutic lies in the direction of hollow glass spheres which hav great crushing strength, cemented together with a silic gel ?

The strength of nylon thread has been exploited i the use of ropes for ship fitting. Can this strength b used for structural purposes in the form of a no corrosive reinforcement ? Whilst in the similar manne glass rope and toughened sheet glass may becon structural materials. In the field of plastics, materia are being developed which soon may be of use to th structural engineer. The hardened resin and glas combination is an engineering material the applicatio of which has hardly yet been touched. It is about on third the weight of steel and two-thirds the weight c aluminium, with a very high strength weight rati Glass reinforced plastics have a high impact strength an are extremely resistant to corrosion.

Another development urgently needed is the extensio of the education of the general contractor, his agent and workpeople in the use of modern techniques. Th costs of building are extremely high and development

Fig. 95

nich might help to reduce these costs should not be tarded by high prices arising from the lack of know-dge and experience of contractors in such methods. ne training courses held by the Cement and Concrete ssociation for foremen and others are doing excellent ork in this direction and should be encouraged by all e means in our power.

In conclusion, I would like to offer my sincere thanks all who have assisted me in the preparation of this ldress by supplying photographs and information. ne British Constructional Steelwork Association, The uminium Development Association, The Cement and ncrete Association, and many engineers and firms are nongst those in the list given below :—

gs. 1-11, 16-20, 24	Soil Mechanics, Ltd.
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58	John W. Poltock, A.R.I.B.A.
61	Ove Arup & Partners, and Cement and Concrete Association.
62, 67, 71, 79-82, 84, 85	Cement & Concrete Association.
63	Turner & Drinkwater and Cement and Concrete Association.
64	The Trussed Concrete Steel Co., Ltd. and Cecil Burns, F.R.I.B.A.
65	Taylor Woodrow Construction, Ltd.
68, 69	E. W. H. Gifford, B.Sc.
70	Stent Precast Concrete, Ltd.
72	Bernard Sunley & Sons, Ltd.
78	Sir Robert McAlpine & Sons, Ltd., and British Electricity Authority.
86, 87	Stewarts & Lloyds, Ltd.
88, 95	Head Wrightson Aluminium, Ltd.
89, 90	Butters Bros. & Co., Ltd.
91	Ove Arup & Partners.
92	Northern Aluminium Co., Ltd.
93	S.M.D. Engineers, Ltd., of Slough, and De Havilland Aircraft Co., Ltd.
94	S.M.D. Engineers, Ltd., of Slough.

Book Reviews

Rigid Frame Formulas, by A. Kleinlogel. (New rk : Frederick Ungar Publishing Co. London : osby Lockwood) $9\frac{1}{2}$ in. × 6 in. pp. xx plus 460. s.

Structural designers who have used Kleinlogel's nous work in one or other of its German editions l need no introduction to this edition in English. r others, a short description will perhaps be of interest. The book is essentially a handbook in which the utions of 114 statically indeterminate frameworks recorded in algebraic form. Practically all the iants of the single-span single-storey portal frames covered, including some with curved members and ne with ties, as well as a number of box frames. A pages are devoted to each frame, so that the bending ment expressions for all the principal modes of loading be given. The results for irregular loadings are en in terms of the freely supported bending moment gram, but even these are worked out in detail for a mber of special cases.

Influence line coefficients are given explicitly in some es ; in others they can be obtained most readily by nbining the formulæ given with those in the com-nion volume " Beam Formulas."

It will be seen that an immense amount of information thus made available, and it is necessarily in a highly ndensed form. The designer must be prepared to ster the symbolism, sign-convention and organisation the book before he can make full use of it, but this sents no real difficulty, in spite of the " European " her than " English " style.

The book has three main uses. The formulæ can be d directly in the design of rigid frames to give a much re rapid solution than can be obtained by moment-tribution, for instance. Secondly, the displacements these frames can be quickly computed from the ding moment diagrams, and finally, the frames can be combined to give more complicated indeterminate structures, whose solutions can be worked out in terms of those given.

The more experienced the designer is in advanced analysis the more value will he get from the use of the handbook ; perhaps it is not unfair to add that the experienced man, in particular, would have appreciated a few pages showing how Professor Kleinlogel obtained his results. J. A. L. M.

Practical Mathematics, Volumes I and II, 3rd Edition, by L. Toft and A. D. D. McKay. (London : Pitman, 1951 and 1952.) pp. 504 and 536, $8\frac{1}{2}$ in. × $5\frac{1}{2}$ in. 20s. and 30s. respectively.

The third edition of this book has been published in two volumes to meet the needs of students preparing for an engineering degree of the University of London, the two volumes dealing with the subject-matter included in Parts I and II of the syllabus in Mathematics.

A considerable amount of new material has been added to this edition. In Volume I an additional chapter has been included on statistics as employed in the engineering industry, and in Volume II there are new chapters on the complex variable, vector and scalar products and partial differential equations and nomograms. Other subjects have received more detailed treatment, particularly the chapters dealing with ordinary differential equations and applied mathematics.

Although primarily intended for degree students, the book will be found useful in advanced classes in technical colleges. The clearly worked examples should be of great help to students and a large number of problems for solution, including many taken from examination papers of the University of London, are included in the book.

Design and Construction of Welded Portal Frame Warehouse Building Designed by the Plastic Method*

By E. J. Callard, B.A., M.I.Mech.E.

Synopsis

This paper briefly describes the design and construction of a simple portal frame building, for which the steel framework was designed by the plastic method. Figures are given to show the economy achieved in the usage of steel over that which might have been used had the framework been designed by elastic methods, and the simple construction of the connections between the rafter and column members is illustrated.

Introduction

Early in 1950 it became necessary to design, as part of an extension of an existing factory, a building to be used for the storage of liquid products packed in drums of 5 gallons capacity. The site area available for its construction was limited, and it was important to achieve maximum utilisation of the storage space to be provided.

After an intensive study of the practicable methods of handling and storing drums it was decided that handling would best be effected by fork lift trucks and timber pallets and storage by stacking drums on pallets, in tiers, each tier being made up of four pallets and each pallet supporting 32 drums. To obtain quick access to any pallet and achieve minimum disturbance of the tiers, tubular racks were designed so that each pallet rested in its own " pigeon hole," and the total weight of all products stored was transmitted through the racks to the building floor. The building structure was thus required to serve the simplest purpose—to provide shelter for its contents from wind, sun and excessively high or low temperatures—and was free of all platforms and other loads. Further, it was apparent that an operating space entirely free of obstruction from roof truss ties such as would be provided by a ridge roof portal frame would be advantageous. Having decided on the type of frame, and in view of the simplicity of the structure and absence of any internal loadings, it proved attractive to us to design the frames by the plastic method. A uniform section all welded pitched portal frame, having fixed bases, was decided upon, and it will be seen later that a considerable economy in steel was obtained.

The project was a strictly commercial one throughout. Some of the design conditions given below, particularly the roof cladding and purlin spacing, were modified after the project had commenced, with the result that the actual steel usage was greater than would have been necessary had full advantage been taken of the reduction in loading occasioned by these changes, and, in fact, had more time been devoted in the design stage to establishing the most economical frame spacings, scantlings, etc.

It is thought, however, that a record of the act design and method of construction adopted in t practical case may be of general interest.

Design Conditions

Maximum utilisation of the space to be provided, a the handling methods to be adopted required a build of 60-ft. span and of minimum free height of 15 whilst the site conditions imposed a maximum length 195 ft. Although the roof cladding used was a lig corrugated resin bonded sheet known as " Corroplas the steelwork was designed to carry corrugated asbes sheets and the frame was of uniform section througho Walls were to be of brickwork 9 inches thick. Th considerations together with the requirements B.S.S. 449 resulted in the following design dimensi and loadings :—

(1)	Fixed base pitched portal frame.	
(2)	Load factor for dead loads ...	1.75
	,, ,, ,, wind loads ...	1.40
(3)	Span	60 ft.
(4)	Height, floor level to eaves ...	15 ft.
(5)	Asbestos sheeted roof carried on angle purlins at 4 ft. 7 in. centres giving a sheeting load of ...	3.5 lb./ft.
(6)	Snow load, equivalent to ...	10.0 lb./ft.
(7)	Self weight of frames, assumed	2.5 lb./ft.
(8)	Weight of purlins	2.0 lb./ft.
(9)	Wind loading equivalent to 60 m.p.h. on side and roof, i.e., wind pressure p, ...	8.2 lb./ft.
(10)	Frame spacing between centres	15.0 ft.

Load Factor

British Standard Specification 449 (Ref. 1) sta that a load factor of 2 shall be employed in the des of fully rigid frames. Since one object of designing structure in accordance with the plastic theory was compare the weight of steel required with that fo structure designed by the elastic theory, the comparis would only be true if in each method the same fac was employed in each case. It was therefore deci at this stage to neglect any limitations due to conside tion of column slenderness and to adopt for the plas method the same dispensations as granted by B.S.S. to the elastic theory, where for a structural steel B.S. 15 : 1948 having a minimum lower yield point 15.25 tons/sq. in. maximum fibre stresses due to bend are limited to 10 tons/sq. in., giving for a rolled st joist with a shape factor of 1.15 a load factor of 1. Further, B.S.S. 449 allows an increase in working stres of 25 per cent. where the stresses result from a combi tion of wind and other loads, so that, for dead plus l plus wind load, a factor of 1.40 was considered accepta for plastic design. Even so, as is more fully argued Baker (Ref. 2), a fully redundant structure would ca an even greater load, since more than one plastic hi

**Paper to be read before the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1, on Thursday, January 28th, 1954, at 6 p.m.*

ıust develop before the structure would collapse, and ıe true load factor is therefore considerably higher.

The load factor achieved on the R.S.J. section selected ɪr fabrication of the frame was 2.26 for the design ɔads. Since the " Corroplast " roof cladding used was

extent that plastic hinges occur and effectively convert the frame into a mechanism. The position and number of plastic hinges necessary to create this condition of collapse are obtained by trial and error, assisted by considerations of the geometry of the frame.

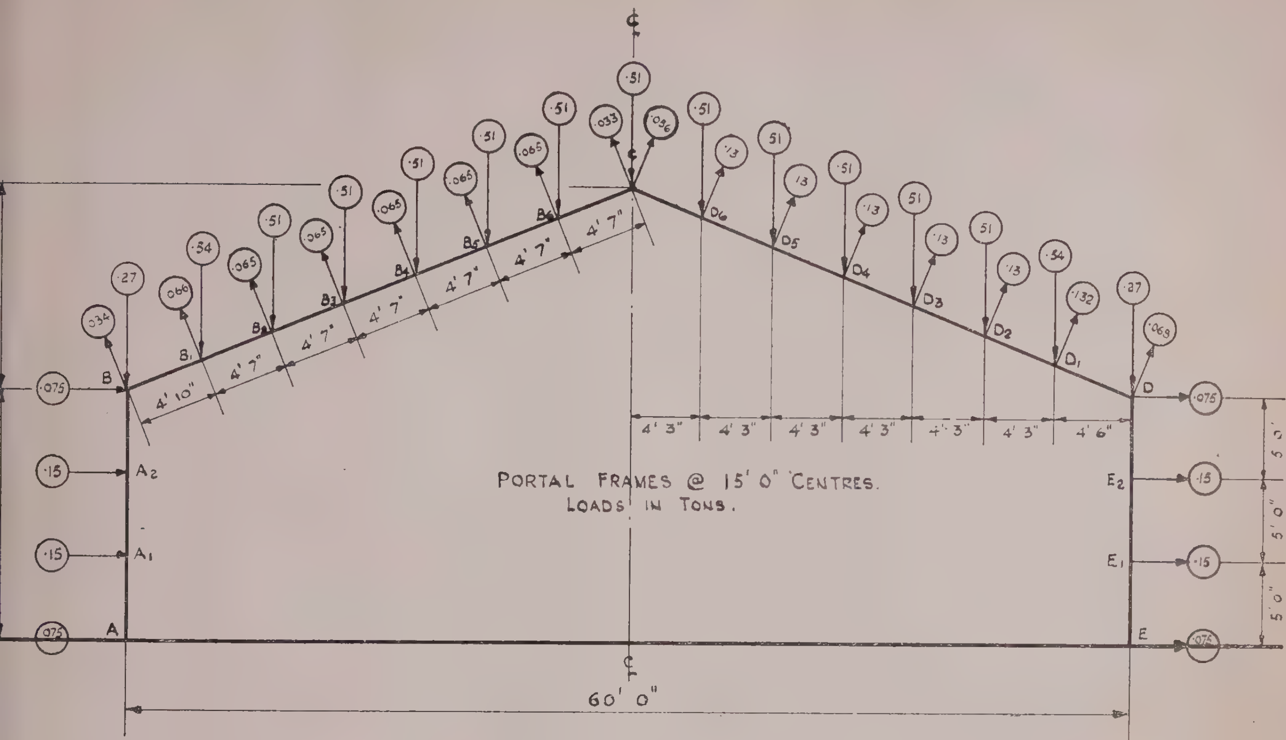

Fig. 1.—Loading on frame

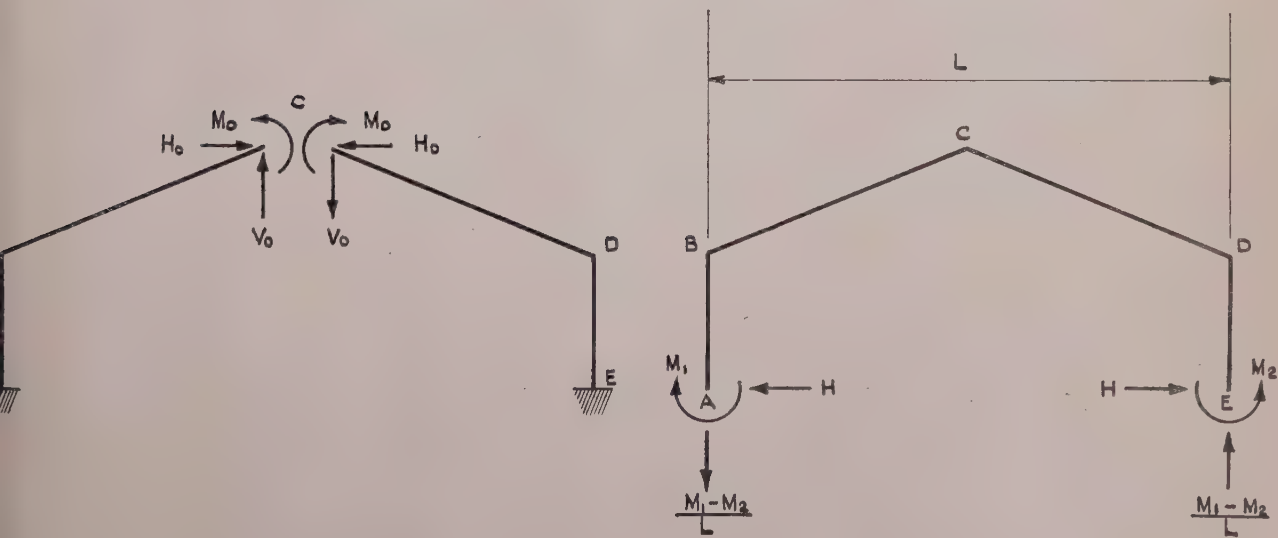

Fig. 2.—Reactions on frame

ıter in weight than asbestos, for which load the ıcture was designed, the load factor on the frames as ıstructed was in excess of 2.26.

Design for Dead and Superimposed Loads

The design conditions given above create the loading tem on the frame shown in Fig. 1.

Considering only the dead and superimposed loads frame will collapse when the loads increase to the

Consider the frame to be split at the apex C into two identical cantilevers ABC and CDE (Fig. 2).

Then for the cantilever A,B,C, the free moments are calculated from the dead and superimposed loads shown in Fig. 1 and are plotted graphically as a,b,c, (Fig. 3). Now the equilibrating forces are acting at A and E (Fig. 2) and give rise to reactant moments due to M_0, H_0, and V_0 at C, or M_1, V_2 and H at A ; further, where plastic hinges occur, the sign of the plastic hinge moment

must be alternatively positive and negative if the frame is ultimately to behave as a mechanism whereby, if one hinge opens, the adjacent hinge must close.

The position of the plastic hinges and the value of the plastic hinge moment M_x, are found by a process of trial and error.

These values give a reactant line at a_3, b_3, c_3, d_3, e at no point in the frame is the plastic hinge mome 15.8 tons ft. exceeded, the actual moments bein difference between the free moment line and the rea moment line. Thus to cause failure, i.e., to conver frame into a mechanism, it is necessary for six hing

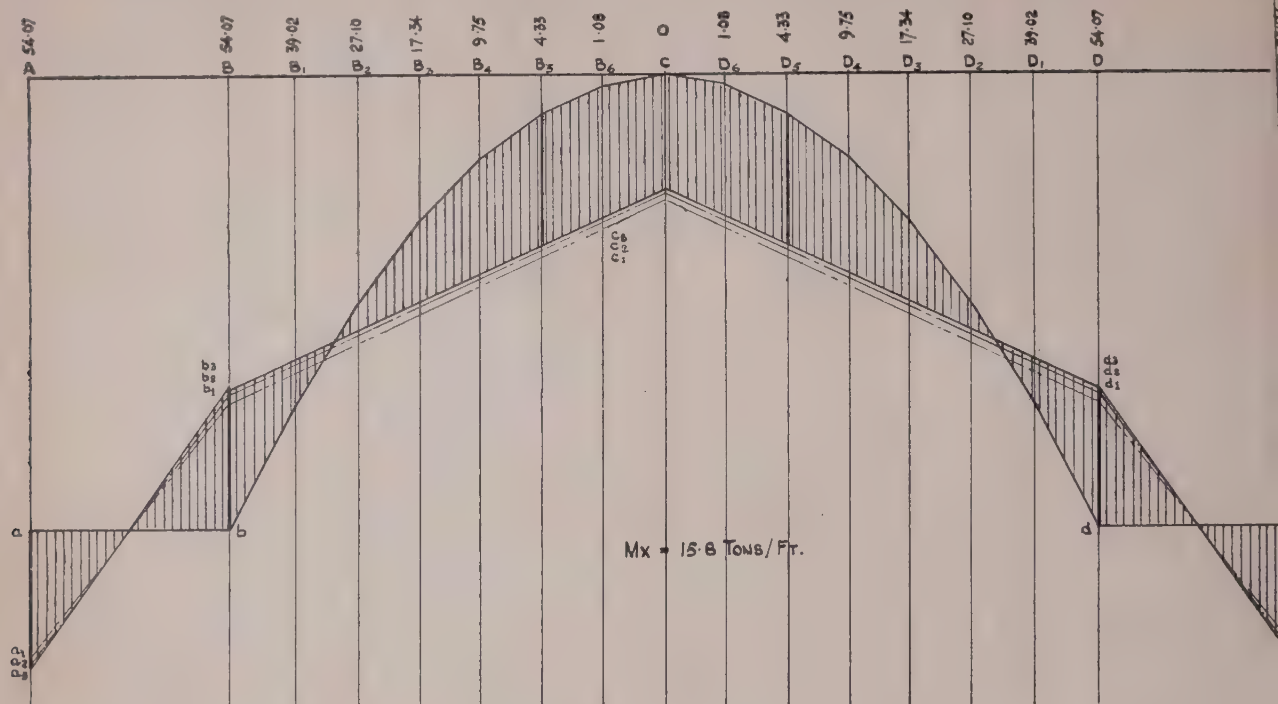

Fig. 3.—Free Moment diagram

Assume plastic hinges occur at A, B, C, D and E. Then the following equations hold :—

At A, assuming a positive moment,

$$-M_1+M_x = -54.07$$

At B, for a negative moment,

$$-(M_1 - 15\,H)+M_x = -54.07$$

At C, for a positive moment,

$$-M_1 = 54.07+M_x$$

Hence $M_x = 15.02$ tons ft.
$M_1 = 69.09$ tons ft.
$H = 2.00$ tons.

Plotting these figures on the free moment diagram (Fig. 3) gives a reactant moment line a_1, b_1, c_1, d_1, e_1 which shows on inspection that the value M_x is exceeded at B_6. This therefore cannot be the correct solution.

Repeating this process, assuming plastic hinges at A, B, B_6, D_6, D and E, rewriting the equations, solving for M_x, M_1, and H, and plotting on the diagram gives a reactant line a_2, b_2, c_2, d_2, e_2 : inspection again shows that the value of M_x found by solution of the equations is exceeded at B_5.

Assuming plastic hinges at A, B, B_5, D_5, D and E, the equations become

At A, for a positive moment, $-M_1+M_x = -54.07$

At B, for a negative moment, $-(M_1-15H)-M_x = -54.07$

and at B_5, for a positive moment, $-(M_1-23.6H)+M_x = -4.33$

Hence $M_x = 15.8$ tons ft.
$M_1 = 69.87$ tons ft.
$H = 2.102$ tons.

occur, at positions A, B, B_5, D_5, D and E. The resul mechanism, and the sign convention of the h moments is shown in Fig. 4.

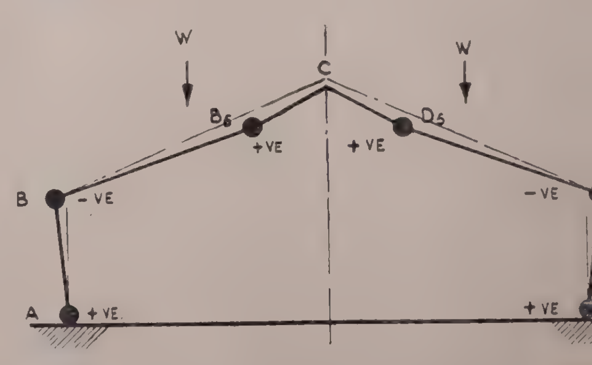

Fig. 4.—Position of " hinges "

Design for Dead, Superimposed and Wind Lo

The wind conditions assumed for design purposes equivalent to the forces shown in Fig. 1, assuming blowing from left to right.

The calculated moments created by these wind f are plotted in the top portion of Fig. 5. These moments, when combined with the free moments, the combined moment diagram *abcde*.

The wind loading on the frame shown in Fig. 6 obviously cause collapse of the frame to occur tow the right, but it is not clear where the hinges will oc Under the unsymmetrical system of loading, howe

ir hinges only need occur before a collapse condition reached.

From inspection of the combined free moment agram, Fig. 5, the free moment at B is nearly equal to e free moment at A, i.e., the portion ab of the moment iagram is almost horizontal. With a negative plastic

At B_6 for a $+^{ve}$ moment $-\left(M_1 - \frac{25.75}{60.0}(M_1 - M_2) - 25.3H\right) + M_x = 0.93$

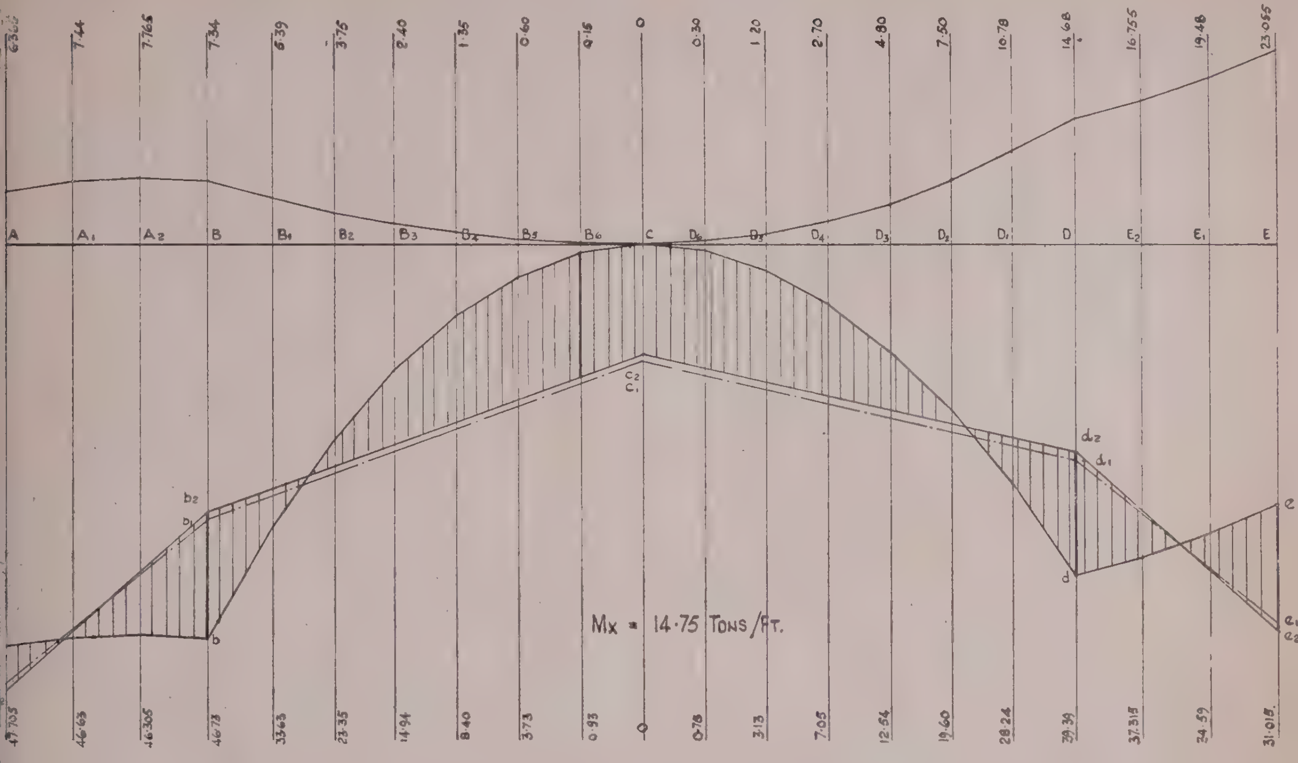

Fig. 5.—Combined Moment diagram

inge moment at A, the reactant moment diagram would equire the slope a_1b_1 to be the same as the slope e_1d_1. ow the line bb_1 at B will always be greater than the inge moment aa_1 at A, and if the reactant moment line $_1b_1$ were to cross the free moment line ab thus altering e sign of the hinge moment at A, the stanchion AB

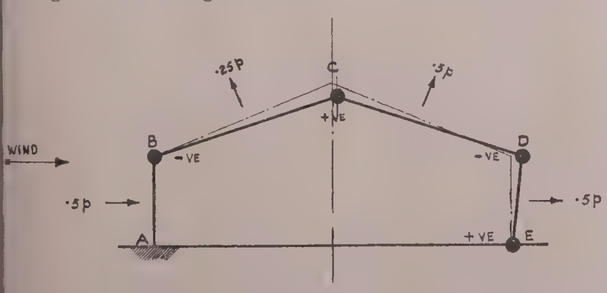

Fig. 6.—Behaviour of frame under wind load

ould collapse against the direction of the wind. It llows that there cannot be a plastic hinge at A without using a greater moment at B and therefore a plastic nge will occur at B and not at A.

The position of the hinges and value of the hinge oments are again found by trial and error. As a st attempt, hinges are assumed at B, C, D and E and e reactant moment line $a_1b_1c_1d_1e_1$ is plotted. Since e reactant moment at B_6 exceeds the value of the hinge oment this cannot be the correct solution. Hinges are erefore assumed at B, B_6, D and E and the solution tained from the following equations :—

B, for a $-^{ve}$ moment $-(M_1 - 15H) - M_x = -46.73$

At D, for a negative moment $-(M_2 - 15H) - M_x = -39.39$

At E, for a positive moment $-M_2 + M_x = -31.02$

Hence $M_2 = 45.77$ tons ft.

$M_1 = 53.11$ tons ft.

$15H = 21.13$ tons.

$M_x = 14.75$ tons ft.

giving the reactant moment line $a_2\,b_2\,c_2\,d_2\,e_2$. Since the hinge moment of 14.75 tons ft. is not exceeded anywhere in the frame, this is the correct solution.

Comparison of Hinge Moments

The plastic hinge moment for dead and superimposed loading is 15.8 tons ft. and that for wind + dead + superimposed loading is 14.75 tons ft. Dead + superimposed loads alone, therefore, provide the greatest moment and the section of the frame must be chosen to provide a resistance of not less than 15.8 × (load factor) tons ft.

Steel Section Required

(a) *Frame*

The required section modulus is given by

$$z = \frac{(\text{Plastic hinge moment}) \times (\text{load factor})}{(\text{Shape factor}) \times (\text{minimum yield stress})}$$

The shape factor is the ratio S/Z where S, the "plastic modulus" is the first moment of area of the section about its centre, and Z is the elastic modulus of the section.

For a standard joist section, $S/Z = 1.15$ (Ref. 2)

Hence

$$z = \frac{15.80 \times 1.75 \times 12}{1.15 \times 15.25}$$

$$= 18.95 \text{ cu. ins.}$$

A rolled steel joist 9 in. × 4 in. × 21 lb. per ft. has a section modulus of 18.03 cu. in., slightly below that required. A 10 in. × 4½ in. × 25 lb. per ft. has a section modulus of 24.74 cu. in. and was the section decided upon for fabrication of the frame. The choice of this section therefore resulted in an increase in the load factor from the minimum of 1.75 regarded as a design condition, to an actual figure of 2.26, which is in excess of the load factor called for in B.S.S. 449. The lighter section could have been adopted if full advantage was taken of the extremely light weight of the "Corroplast" corrugated roof sheeting rather than designing for the weight of asbestos sheets. Alternatively, a reduction in the space between frames could have reduced the frame section and effected some economy in foundation design.

(b) Purlins

The purlins were designed by the elastic theory and to the requirements of B.S.S. 449, clause 55a. They were designed to be bolted to cleats, the cleats being welded to the portal frame. The effect on purlin weight of designing them by the plastic method was not investigated.

Design of Joints at Apex and Eaves

Bearing in mind that the joints at apex and eaves were to be site welded, a design was required which, whilst preserving the full continuity of the frame in bending and giving adequate stiffness, would obviate the need for butt welds and would make the preparation for welding as simple as possible. No attempt was made to economise in steel by the use of knee braces at the eaves. The design of joint chosen required only that the main joists should be cut in the fabricating shops at the required angle and a flat rectangular plate fillet welded to one end. This plate was of ¾ in. thickness and was a web plate only. This permitted the plate to be fillet welded both to the web and the flange of the joist in the fabricating shops and limited site welding to a fillet weld between the web of the meeting joist and the plate and a fillet weld between the joist flange and the plate. By making the plate a web plate only, the welds were open, giving good access for the electrode, at the expense of requiring a considerable number of welds to make up the joints between the flanges of the joists. The site welding was carried out whilst the joists were held in specially prepared jigs, as illustrated in the photographs, Figs. 9 and 10. Fig. 10 also serves to illustrate the method of welding the column ties. Since the drilling of holes in the frame was to be avoided, a supporting plate for the tie was shop-welded to the stanchion joist, this plate being drilled to take two bolts to locate the eaves tie on site. After the eaves ties had been erected and located by the two bolts a plate was welded to the stanchions immediately above the tie, and the tie then welded to both top and bottom fixing plates, as shown in Fig. 12. This design, whilst serving to avoid the drilling of the frame and giving simplicity in welding, also served to stiffen the joists at the top of the columns. No other web stiffeners were employed, but to give some additional stiffness at the ridge, diaphragm plates were bolted to the ridge purlins at positions on the centre line of each bay.

Column Slenderness

In order to obtain full fixity of the column bases and for convenience in transmitting part of the wall load to the foundations, the columns were constructed 17 ft. in length, making the underside of the column base plates 2 ft. below the finished floor level. This resulted in a slender column, having a slenderness ratio, taking into account the restraint of the 6 in. × 3 in. tie member, of 160. To provide lateral stability during construction the columns of the first and second frames were diagonally braced both above and below a horizontal member. These braces were later removed as wall brickwork proceeded, the brickwork serving to give adequate lateral stability.

Steel usage—Comparison with Elastic Design Method

The total weight of steel in each portal frame designed as explained is 1.33 tons, giving a total steel usage for the fourteen frames of 18.6 tons, and it is interesting to compare the tonnage of steel which would have been employed had the frame been designed by the normal methods. A portal frame, pin jointed at the bases, designed by the elastic theory, required 15 in. × 5 in. × 42 lb. per ft. rolled steel joists, the weight per frame being 2.03 tons and the total weight of steel in the fourteen frames 28.4 tons. The economy of steel resulting from the plastic design compared with that which would have been used in the same building had the original preliminary design by normal methods been proceeded with, was 9.8 tons, or 34.5 per cent. Further economy in steel usage could have been achieved by the plastic method had the frames been designed to take full advantage of the extreme light weight of the roof cladding employed, when 9 in. × 4 in. × 21 lb. per ft. joists could have replaced the 10 in. × 4½ in. × 25 lb. per ft. joists for the frames, giving a total weight of steel of 15.82 tons.

The benefits accruing from this change to a lighter roof cladding were, however, not exploited, as it was felt that owing to the possibility of a change in the future use of the building it might become necessary to provide for additional roof insulation or other changes for which it was desirable to have some reserve of strength in the main structure.

Purlins and column ties accounted for 14.35 tons, so that the total steel employed in the building as erected, excluding reinforcement steel in the foundations and floors, was 32.95 tons.

Foundation Design

The design conditions and reactant moments given above suggest that the foundation for the portal frame should be designed to resist the following conditions :—

(1) Axial loading of 6 tons.
(2) Horizontal load of 2 tons.
(3) An overturning moment of 27.65 tons ft.

It is obvious that (3) above is the condition determining the foundation design.

However, the selection of a 10 in. × 4½ in. × 25 lb. per ft. rolled steel joist for the portal frame, having a section modulus of 24.47 cu. in. where the calculated requirement was a section modulus of only 18.95 cu. in. makes a modification necessary to condition (3). The 10 in. × 4½ in. joist is capable of resisting a full plastic moment of 35.7 tons ft., so that if the foundations are to have the same degree of stability as the steel frame

they must also be capable of resisting the full moment which the frame is capable of exerting upon it, i.e., 35·7 tons ft.

The foundation design adopted was a rectangular section block, the foot of the column being placed eccentrically upon it, and held by six $1\frac{1}{8}$ in. diameter 12 in.-long foundation bolts. In order to resist the tendency for the foundation to spread under the horizontal forces acting upon it, reinforcement was introduced to tie the foundation to the concrete floor of the building and also to the concrete wall footings. After erection of the frame the base was concreted in to sub-floor level. Fig. 7 shows the completed foundation with the column erected upon it, this illustration also serving to show the construction of the column base.

Fabrication and Shop Welding

All steel sections were specified to conform to B.S.S. 15 and test certificates were obtained from the rolling mills for each consignment employed, the sections being supplied from two mills. For one consignment the ultimate tensile strength was 32.8 tons/in.^{2} and elongation 23 per cent. and for the other the ultimate strength was 30.5 tons/in.^{2} and elongation 30 per cent.

Fig. 7.—Column base and foundation

To assist in obtaining accuracy in the mating of the joints to be welded the joists were all saw cut ; no flame cutting was employed.

In conjunction with the steel fabricators and the suppliers of welding rods great care was taken to select electrodes which would give adequate penetration at the root of the weld and also result in a weld deposition which was fully ductile. The detailed design of the joints was such that almost all welds were fillet welds and as far as practicable all welding was carried out in the down-hand position. Further, all welders to be employed in the fabrication were made to carry out welding tests using the same type and size of electrodes to be employed in fabrication of the frame, the welding being carried out in the open air in the same conditions as those likely to apply when the time came to carry out the site welding. The test plates were machined and tensile and bend tests carried out before each welder was passed for work on the frame. Finally, a mock-up of an eaves joint was made and welded so that the welder to be employed could develop a satisfactory technique prior to commencement of welding on the actual frames. Where it was necessary to weld between the insertion plate and the joist at eaves and apex levels the weld consisted of one run of 10 s.w.g. electrode, followed by two runs of 8 s.w.g. using a Unitrode electrode. Provided the welding rod was held nearly vertical and the arc kept as short as possible thoroughly satisfactory welds were obtained. For ancillary welds, such as the purlin cleats and stiffener plates for the

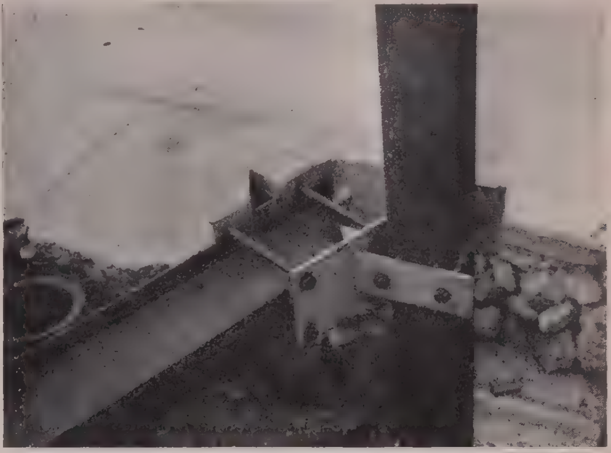

Fig. 8.—Jig for location of rafter during fitting of apex joint

horizontal tie beams just below the eaves, Murex Fastex Five electrodes were used. All welding other than the joints at eaves and apex and the final attachment of the eaves tie beams was carried out in the shop, thus keeping to a minimum the welds to be carried out on site.

Method of Erection

The method of erection adopted was influenced by availability of equipment, the limited access to the site and the wish to keep down the cost of erection to a minimum consistent with an adequate standard of dimensional accuracy and soundness of welding.

Having completed the site preparation work and constructed the column foundations, the columns with

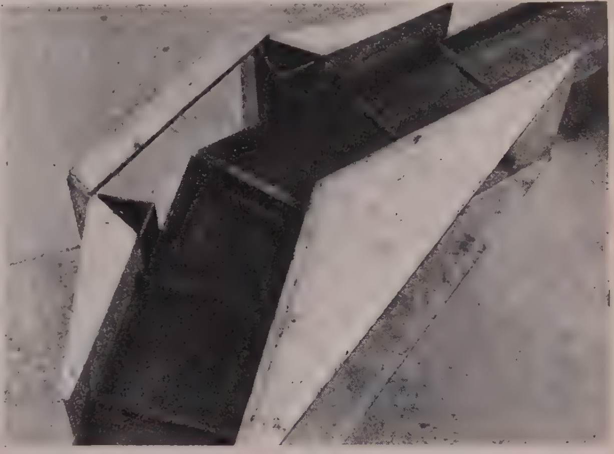

Fig. 9.—Cap jig for apex joint

the foundation bolts suspended in the base plate were lowered on to the foundation. The line of columns along one side of the building was then grouted in, great care being taken to line them up, the bolts being tightened up on the base plate after the grout had hardened. Alignment was checked by stretching piano wires from

one end of the building to the other, one wire at the head and the other at the foot of the columns. The columns on the other side of the building were also very carefully positioned and bolted down ; they were not, however, grouted in so that slight horizontal movement

Fig. 10.—Jig for welding eaves joint

could be provided should adjustments become necessary to obtain a good fit of the rafters to the tops of the columns before welding of the eaves joints commenced. Having obtained accurate alignment and positioning of the columns on either side it was possible to use the columns to assist in obtaining accurate fit of the rafter members and so avoid a jig, which might have been large and unwieldy, to check the accuracy of the span of the rafter members. The special jig for location of the end rafter member during fitting of the apex joint is shown in Fig. 8. The two rafter members were laid out on the ground on adjustable stools and one end of each member was held by the jig (Fig. 8), thus locating the end of the rafter whilst the fit of the apex joint was checked, the rafters during this process lying in the

Fig. 11.—Interior view of completed building

horizontal plane. For the apex joint a cap jig of the form shown in Fig. 9 was fitted, and whilst still in the horizontal plane the joint between the insertion plate and the profile of the rafter was carefully examined to see that the welding gap was uniform and of permissible dimensions. A maximum gap of 1/16 in. was specified and to obtain this condition it was found necessary in some cases to grind the end of the joist to fit the junction plate. No joint was passed for welding until the maximum gap between the two parts to be welded was 1/16 in. Whilst the rafter members were rigidly held between jigs in the horizontal plane, the first welds were carried out on the apex joint. Using a 10 s.w.g. Unitrode electrode the vertical down-hand weld was made between the joist flange and the insertion plate and then turning along the web in a down-hand position, finishing in a vertical run along the other flange. On completion of this first run of welding, additional ties, turning the rafter members temporarily into an A frame, were bolted to the purlin cleats, so permitting the frame to be turned over after first releasing the jigs (Fig. 8) holding the rafters to the columns. Having turned the rafter members over, the free ends were attached by means of the jigs to the columns of the next frame and the first run of weld laid down. This weld was followed by two runs of 8 s.w.g. Unitrode electrode to complete the weld section on one side of the apex joint. The frame was then again turned over, attached by the jigs to the first pair of columns, and two runs of 8 s.w.g. Unitrode were laid down to complete the welding on that side of the section.

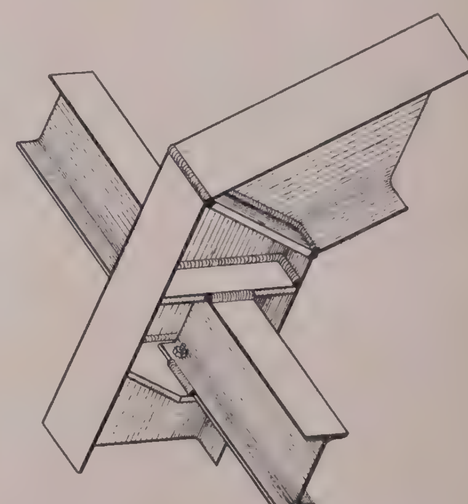

Fig. 12.—Design of eaves joint and tie fixing

Having released the rafter from the column jigs the frame was then turned so that its plane was in a vertical position with the apex downwards, thus allowing the butt weld between the joist flanges to be completed, this weld being made with three runs of 8 s.w.g. Unitrode electrode. The frame was then lifted into a vertical position with apex upwards, allowing the open butt weld at the top of the apex joint to be made in a down-hand position, using two runs of 8 s.w.g. electrode. Throughout the welding work very little porosity was detected. It was, however, sometimes found at the beginning and end of a run, perhaps due to the change in speed of welding at these points. Each run of weld was carefully inspected and in the event of any evidence of porosity the doubtful section was chipped out and re-welded. The frame was then ready to lift on to the columns after detaching the locating jigs (Fig. 8).

Jigs as shown in Fig. 10 were then fitted to the top of the columns and the rafter members lifted into position and clamped into the jigs. The fit of the joints was then carefully examined and checked, only minor adjustments being necessary to achieve the specified standard of fit by grinding the end of the rafter members. A temporary knee brace was clamped to the rafter and column members and whilst welding was in progress the weight

of the rafter members was taken by lifting gear and the entire portal frame braced so that it was in the strictly vertical position. The welding of the eaves joint then followed in a similar manner to that adopted for the apex joints, thorough inspection being made between each run. The 5/16 in. section welds between the joists and the insertion plate were made, making one run of 10 s.w.g. and finishing with two runs of 8 s.w.g. Unitrode electrode. The more open butt joint at the upper flanges of the joists was then made, eleven runs of 8 s.w.g. being necessary to fill this joint due to the open nature of the joint and the fact that the junction plates were effectively web plates only. The lower butt joint was made by four runs of 8 gauge electrode.

During the welding of the joints of the first frame, the second frame was being laid out in readiness for the welding of the apex joint, and the fit of the members checked. Welding and erection of subsequent frames proceeded smoothly and the rate of progress of one frame per day was ultimately achieved, using one welder only on the joints. The total welding time employed during construction was only 27½ man days in spite of the fact that the frequent checks made on dimensional accuracy, and the amount of handling of the members required, led to less continuous employment of the welder than is usual on site welding work.

Mention has been made of the need to grind the ends of the members to achieve a uniform gap at the point of welding. The extent to which this work was necessary, although not large, was greater than expected and was due to the slight distortion of the junction plates which had occurred in shop welding. It is considered that the amount of this work would be greatly reduced on any subsequent job of this nature by a modification in procedure, whereby the fixing of the junction plates was not completely done in the shops and the welding technique adjusted to avoid distortion.

The columns of the first and second frames were diagonally braced to provide lateral stability although, as has been explained above, these braces were removed when the walls were constructed. Also, it was felt desirable to fit diagonal braces between the first and second rafter girders at each end of the building to obtain initial rigidity and to assist in positioning the end frames of the building. Although these were intended as temporary members until sufficient purlins could be fixed following erection of the third frame, they were not removed when construction was completed. The purlins were designed to span two bays, so that short purlins only could be fixed when the second frame was erected. As successive frames were erected the purlins were used to provide adequate bracing of the frames during the welding of the eaves joints.

Cladding

When fixing of the purlins had been completed the remaining line of columns was grouted in and the upper part of the foundation poured to make the frames fully encastre at the feet. The roof covering of "Corroplast," containing approximately 5 per cent. of the area in Perspex, was erected in the usual way. Although the use of Corroplast sheeting instead of asbestos reduced the weight of the roof covering by 2½ lb. per sq. ft., it was felt desirable to place the roof covering before bricking up the walls in order to allow the frame to take up the deflection resulting from the weight of the roofing prior to the construction of the brickwork. Bricking up of the walls proceeded immediately following the completion of the roof covering, the brickwork being 9 in. thick throughout and serving to give the columns adequate lateral stiffness. Further, the columns were protected by encasing in concrete. The completed structure, shown in Fig. 11, after installation of heating and lighting equipment was ready for occupation in February, 1952.

Acknowledgements

The author wishes to acknowledge the assistance given with this project by his colleagues in the Engineering Department of the Paints Division of I.C.I. Ltd., by Professor J. F. Baker and Dr. J. W. Roderick, of Cambridge University, during the design stage, and thanks Mr. H. Knott, of Croggon & Co., Ltd., to whom the design of the jigs is due, for his interest and assistance in all design and construction work.

References

[1]B.S.S. 449 : 48 "The Use of Structural Steel in Building."
[2]Baker, J. F. "The Design of Steel Frames," THE STRUCTURAL ENGINEER, October, 1949.

Book Review

Advanced National Certificate Mathematics, Vol. I, by J. Pedoe. (English Universities Press, 1952.) 8½ in. × 5½ in. 346 pp. 15s.

The contents are intended to cover the requirements of a normal Higher National Certificate syllabus : a limited revision of the trigonometry, algebra and calculus of the Ordinary Grade syllabus ; partial fractions, differentiation, co-ordinate geometry, integration and its applications, infinite series, partial differentiation, complex numbers and hyperbolic functions. There is a chapter on elementary differential equations which includes an explanation of the mathematical treatment of the bending of beams under both continuous and discontinuous loading, and also Euler's theory of struts. An interesting feature is a chapter on statistics.

A few suggestions might be made for future editions ; an index would add to the value of the work ; the treatment of methods of integration might be somewhat more systematic ; the differentiation of an implicit function is worthy of more space, whilst the omission of Taylor's Theorem is a pity, as many Higher National Certificate syllabuses include it.

However, a generally favourable impression is obtained, and special mention should be made of several good points : the provision of a separate chapter on partial differentiation ; the suitability of the material chosen for the co-ordinate geometry sections ; and explanatory notes which are very helpful—*e.g.*, those on page 250 on the Parallel Axis Theorem. The worked examples are well chosen, and the examples at the end of the sections are well graded, some of the end ones being of considerable difficulty. Finally, there is a set of about 100 miscellaneous examination questions, with answers, taken from assessed Higher National Certificate papers, and from the B.Sc.(London) Engineering examination, Part 1.

This work can be confidently recommended for Higher National Certificate students ; it is very suitable for self-study and any intelligent student should profit by it.

E. S. G.

Institution Notices and Proceedings

GENERAL MEETING

A General Meeting of the Institution of Structural Engineers was held at 11, Upper Belgrave Street, London, S.W.1, on Thursday, October 1st, at 6.0 p.m., when the Presidential Address for the Session 1953-54 was given by Lt.-Colonel R. F. Galbraith, M.C., B.Sc., M.I.Struct.E., A.M.I.C.E.

The Chair was taken by the retiring President, Mr. Ernest Granter, B.Sc., M.I.C.E., M.I.Struct.E., who welcomed the guests, and presented the following medals for the Sessions 1950-51 and 1951-52 :—

INSTITUTION BRONZE MEDAL (1950-51) to Mr. A. Goldstein, for a paper presented jointly with Lt.-Colonel G. W. Kirkland, M.B.E. entitled "Design and Construction of a Large Span Prestressed Concrete Shell Roof." Colonel Kirkland received his medal last October, but Mr. Goldstein was at that time abroad, and the presentation of his medal was deferred until his return.

INSTITUTION SILVER MEDAL (1951-52) for the best paper read before the Institution during the Session 1951-52, to Mr. F. R. Bullen, for a paper entitled "Unusual Design for a Large Constructional Shop."

INSTITUTION GOLD MEDAL (1951-52) to Professor J. F. Baker, O.B.E., for his services to the Institution and to structural engineering science over a long period in connection with the Steel Structures Research Committee of the Department of Scientific and Industrial Research, and his continued interest and research into problems of structural design as envisaged by the Plastic Theory.

After some introductory remarks regarding Colonel Galbraith's career in connection with the Institution, the Chairman invested him with the Presidential Badge.

Colonel Galbraith then took the Chair and called upon Mr. Stanley Vaughan, Vice-President, and Mr. Hugh Davies to propose and second a vote of thanks to Mr. Ernest Granter, for his work as President of the Institution during the Session 1952-53.

The vote of thanks was carried, and Colonel Galbraith then gave the Presidential Address for the Session 1953-54, which is printed in this issue.

At the conclusion of the Address a vote of thanks to the President was proposed by Mr. J. Guthrie Brown, and seconded by Mr. F. R. Bullen. This was carried with acclamation, and the proceedings then terminated.

SPECIAL GENERAL MEETING

A Special General Meeting of the Institution of Structural Engineers was held at 11, Upper Belgrave Street, London, S.W.1, on Friday, October 9th, at 6 p.m., for the purpose of effecting, conditionally upon the approval of Her Majesty's Privy Council thereon, the alterations and additions to the Bye-Laws as set out in a resolution which was submitted. The President, Lt.-Colonel R. F. Galbraith, M.C., B.Sc., M.I.Struct.E., A.M.I.C.E., was in the Chair.

The Assistant Secretary, Mr. Herbert R. H. Gray, A.C.I.S., read the notice convening the meeting which contained the resolution.

The President invited members to comment on the proposals before the meeting.

After discussion the proposals were put to the meeting and carried.

ORDINARY GENERAL MEETING

An Ordinary General Meeting of the Institution of Structural Engineers was held at 11, Upper Belgrave Street, London, S.W.1, on Thursday, November 26th, 1953, at 5.55 p.m., Lt.-Colonel R. F. Galbraith, M.C., B.Sc., M.I.Struct.E., A.M.I.C.E. (President), in the Chair.

The following members were elected in accordance with the Bye-Laws. Will members kindly note that the elections, as tabulated below, should be referred to when consulting the Year Book for evidence of membership ?

STUDENTS

BUTLER, Frank Marchant, of Johannesburg, South Africa.
FLETCHER, Kenneth Albert, of Bexleyheath, Kent.
FONG WENG SENN, of Singapore.
FORREST, Esli James, of Papatoetoe, Auckland, New Zealand.
FORSYTH, James Kerr, of Woking, Surrey.
FOWLDS, Raymond Barry, of Port Elizabeth, South Africa.
GASIOROWSKI, Waclaw, of London.
GILES, Harry Joseph, of Dawlay, Shropshire.
HIGSON-SMITH, David John, of Salisbury, Southern Rhodesia.
JACKSON, Peter, of Newcastle upon Tyne.
KERNS, Donald Patrick, of Durban, South Africa.
KORBUSZ, Tadeusz, of London.
MANN, Edward Alexander, of Thornton Heath, Surrey.
MARTIN, Francois Regis, of Mauritius, Indian Ocean.
NELSON, Kenneth, of Bolton, Lancashire.
NG PING KIN, of Urbana, Illinois, U.S.A.
PANG KIA CHOON, of Singapore.
PATTISON, Roland Derek, of Newcastle upon Tyne, 5.
SKIBA, Henryk, of London.
THOMPSON, Ralph James, of Dunedin, New Zealand.
WATSON, Fred, of Manchester, 18.
YOONG SIEW KIN, of Ipoh, Malaya.

GRADUATES

ABRAHAM, Eapen, B.E. Calcutta, of Calcutta, India.
ADAMS, Wilfred, of Liverpool.
BAILEY, Alan Hale, of Kingston upon Thames, Surrey.
BAILEY, John Clifford, B.Sc.(Eng.) London, of Oakengates, Shropshire.
BASU, Amar Krishna, of London.
BEIMEL, Chaiym, B.Sc.(Eng.) Cape Town, of Cape Town, South Africa.
BERRY, Derek Wilfred, of Reading.
BIRD, Erik Hugh, B.Sc.(Eng.) London, of Watford, Herts.
BRITT, Geoffrey Brian, M.A.(Cantab.), of Birmingham.
BROWN, Jack Henry, of Peterborough.
BUNNEY, Dennis Albert, of London.
CAMERON, Archibald Wilson, of London.
CARDEN, Thomas William Arthur, of London.
CHAR, Sreenivas Ranga, of Jogeshwari, Greater Bombay, India.
COSGRAVE, James, of London.
COURT, Robert, of Feltham, Middlesex.
DAVIES, Gwilym Morris, of Maesteg, Glamorgan.
DEADY, Patrick Francis, B.E.(Civil) Dublin, of Rathmines, Dublin.

D'Sousa, Anthony Victor, B.E.(Civil) Bombay, of Bombay, India.
Easterbrook, Peter Lawrence, B.Sc.(Wales), of Montreal, Canada.
Elion, Allan Anthony, B.Sc., of London.
Garbutt, Alan Watson, of Liverpool.
Goldstein, Antony Edward, B.Sc.(Civil) Rand, of Johannesburg, South Africa.
Hale, Kenneth Reece, of Liverpool.
Hammett, Michael John Redwood, of Billericay, Essex.
Jelley, Edward Leslie, B.Sc.(Eng.) London, of Romford, Essex.
Kalim, Syed Mahbubul, B.E.(Civil) Calcutta, of Derby.
Kanjee, Sadroodeen Abdulla Alibhai, of London.
Kathiramalainathan, Kathirgamar Veeragathy Sinnathurai, B.Sc.(Eng.) London, of London.
Key, David Edwin, B.Sc.(Eng.) London, of London.
Khalafalla, Ahmed Omer, of London.
Khan, Saeed Ahmad, of Karachi, Pakistan.
Klug, Benjamin, B.Sc.(Civil) Natal, of London.
Lau Foo Sun, B.Sc.(Eng.) London, of London.
Lawton, David Alwyn Arthur, of Ogmore-by-Sea, Glamorgan.
Levy, Musa Sassoon, of London.
Ludlam, Donald Thomas, of Salford.
Mallick, Prodyot Kumar, B.E.(Civil) Calcutta, D.I.C., of London.
Miller, Maurice, of Shepperton, Middlesex.
Minch, David Berchmans, B.A.I.(Dublin), of London.
Musson, Alan, of East Kirkby, Notts.
Neish, Robert Donald, of Edinburgh.
Norris, Peter Richard, B.Sc.(Eng.) Natal, of Durban, South Africa.
Owens, Kenneth Alwyn, B.Sc.(Tech.) Manchester, of London.
Owens, Owen, B.Sc.(Eng.) Rand, of Livingstone, Northern Rhodesia.
Pickering, Fred, of Middlesbrough.
Robinson, William James, of Stockport, Cheshire.
Shakur, Abdul, B.Sc.(Eng.) Punjab, of Lahore, Pakistan.
Silgardo, Mervyn Bernard, B.Sc.(Tech.) Manchester, of Rangoon, Burma.
Tate, Adrian Peter Kisse, B.Sc.(Eng.) London, of London.
Tate, Geoffrey William, of Flixton, near Manchester.
Tendolkar, Prabhakar Shamrao, B.E.(Civil) Bombay, of Tallington, near Stamford, Lincs.
Whittle, James Raymond, of Gateshead, Co. Durham.
Wilson, Gordon Frank, of Birmingham.
Wilson, John Fleming, B.Sc.(Eng.) London, D.I.C., of Wallington, Surrey.
Wood, Frederick George, A.M.I.C.E., A.M.I.Mun.E., of Lytham St. Annes, Lancs.
Zelman, Maier, B.Sc.(Tech.) Manchester, of Seven Kings, Ilford, Essex.

Transfers

Students to Graduates

Attwood, Roy Leonard, of Derby.
Blow, Leslie William Furse, of London.
Bradshaw, John Richard, of Grange, West Kirby, Cheshire.
Burtenshaw, Raymond Vincent, of Brighton.
Butler, Alan, of Manchester.
Hones, Dennis, of London.
Kenny, Alphonsus Jerome, of Wallasey, Cheshire.
Kucieba, Jerzy Stefan, of London.
Lawson, Desmond Kane, B.Sc.(Civil) Belfast, of Trentham, Hutt Valley, New Zealand.
McKenzie, Alexander Stewart, of Salford, Lancs.
Marriage, Eric Francis, of Enfield, Middlesex.
Perry, Leonard Ernest Arthur, of London.
Tobin, John Raymond, of Wallasey, Cheshire.
Wallace, David John, of Birmingham.

Graduates to Associate-Members

Ballard, Eric Hugh, B.Sc.(Civil) Belfast, of Enfield, Middlesex.
Beck, Roy Cecil, of Norwich, Norfolk.
Brett, George Edmund, of Middlesbrough, Yorks.
Carolan, Edward Joseph, B.E., N.U.I., of Dublin.
Conway, Gerald Ernest, of Welling, Kent.
Davies, Brian Clifford, of Selsdon, South Croydon, Surrey.
Deshmukh, Madhukar Dattatray, B.E.(Civil) Bombay, of Nasik City, Bombay State, India.
Fasehun, Ebenezer Olawanle Oladipo, of London.
Halsall, John Denys, of Salford, Lancs.
Hill, Peter Henry, of London.
Jenkinson, Anthony Richard, of Egham, Surrey.
Kerr, Gordon Cecil, of London.
L'Oste-Brown, Anthony Joseph, of Burton upon Trent, Staffs.
Maclean, Alexander James, of Liverpool.
Nandi, Subir, B.E.(Civil) Calcutta, of London.
Purvis, John Ferguson, of Guisborough, Yorks.
Richardson, Ronald Edward, of London.
Rogers, Kenneth Jordan, of Bretby, near Burton upon Trent.
Rose, Gordon Michael, of Toronto, Ontario, Canada.
Rowland, Vernon Roy, B.Sc.(Civil) Bristol, of Bath, Somerset.
Shrotri, Ganesh Shankar, B.E., of Poona, India.
Smith, Reginald George, B.Sc.(Hons.) Edinburgh, of Vancouver, B.C., Canada.
Stock, David Athol, of Christchurch, New Zealand.
Sukhyanga, Sommataya, of Bangkok, Thailand.

Associate-Members to Members

Brickell, Richard Goulden, M.I.C.E., of Wellington, New Zealand.
Dobson, Leslie, of Gosforth, Newcastle upon Tyne, 3.
Edwards, Wilfred Price, of Wellington, New Zealand.
Faber, John Gordon, B.Sc.(Eng.), London, A.M.I.C.E., A.C.G.I., A.M.Am.Soc.C.E., of Harpenden, Herts.
Reem, Herbert Felix, C.E., A.M.Am.Soc.C.E., of New York, U.S.A.
Rowland, Evan Hugh, of Salisbury, Southern Rhodesia.
Van Onlangs, Johannes, of Durban, South Africa.
Ward, Leonard Edwin, of Stoke Poges, Bucks.
Wong Tai Cho, B.Sc.(Eng.) Hong Kong, of Hong Kong.

Re-Admissions

Member

Ribbins, William Vernon, of South Wigston, Leicester.

Associate-Member

Chau Iu-Nin, B.Sc.(Eng.) Hong Kong, of Hong Kong.

OBITUARY

The Council regret to announce the deaths of Colonel Peter James Bowling, John MacDermid, Professor Parm Anand Midha, Donald Ross, William Harley Saxton (Members); James McGregor, D.S.O., David MacIntosh Robertson (Retired Members); John Gilbert Pitney Meaden (Associate); David Barrie Marrs (Associate-Member); Jan Treichel (Graduate).

EXAMINATION PRIZE LIST, JULY, 1953

The Council have awarded the following prizes in connection with the examinations held in July, 1953 :—

ANDREWS PRIZE (For the candidate who obtains the highest aggregate of marks in the Associate-Membership Examination, passing in all subjects)

Frank Edgar LEWIS, of London.

HUSBAND PRIZE (For the candidate who takes the whole of the Associate-Membership Examination, passes in all subjects, and obtains the highest marks in the paper, "Structural Engineering Design and Drawing")

Derek BOND, of Bristol.

WALLACE PREMIUM (SENIOR) (For the candidate who takes the whole of the Associate-Membership Examination, passes in all subjects, and obtains the highest marks in the paper, "Theory of Structures ("Advanced."))

Frank Edgar LEWIS, of London.

WALLACE PREMIUM (JUNIOR) (For the most successful candidate in the Graduateship Examination, passing in all subjects)

CHENG HON KWAN, of Hong Kong.

EXAMINATIONS—JANUARY, 1954

The Examinations of the Institution will next be held at centres in the United Kingdom and Oversea on January 5th and 6th (Graduateship), and January 7th and 8th, 1954 (Associate-Membership).

REPRESENTATION

The Council have made the following nominations of members to represent the Institution :—

CITY AND GUILDS OF LONDON INSTITUTE, Exploratory Committee on Concrete Technology :—Mr. D. A. G. Reid (Associate-Member).

CITY OF BIRMINGHAM EDUCATION DEPARTMENT, Building Professional Courses Advisory Committee :—Mr. E. R. Deeley (Associate-Member).

BRITISH STANDARDS INSTITUTION, Technical Committee, Glossary of Terms for Concrete and Reinforced Concrete :—Dr. A. R. Collins (Member).

PUBLICATION WITHDRAWN

Stocks of the following publication are now exhausted and the Report has been withdrawn :—

21/37—Report for the Guidance of Structural Engineers when using High Alumina Cement.

JOURNAL CASES AND BINDING, 1953

A binding case can be supplied for the twelve issues of the Journal, January-December, 1953 (Volume 31), price 11s. 6d., post free. The price for binding volumes is 27s. per volume, inclusive. This is for the half-leather binding which has been in use for some years.

It is requested that all parcels and Journals forwarded for binding should bear the name, address and rank of the member concerned. All volumes for binding must be despatched to the Institution by March 31st, 1954.

An Index will be included in all volumes bound. This Index will not be generally distributed, but members and others wishing to have a copy should apply to the Secretary.

FORTHCOMING MEETINGS

Special Notice. It has been necessary to make a number of changes in the Sessional Programme as printed. The revised Programme of Meetings to be held at 11, Upper Belgrave Street, London, S.W.1, during January, February and March is as follows :—

Thursday, January 28th, 1954

Ordinary General Meeting, 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when Mr. E. J. Callard, M.A., M.I.Mech.E., will give a paper on "Design and Construction of a Welded Portal Frame Warehouse Building Designed by the Plastic Method."

Thursday, February 25th, 1954

Ordinary General Meeting 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when Mr. R. A. Sefton Jenkins, B.Sc., A.M.I.C.E., A.M.I.Struct.E., A.C.G.I., will give a paper on "Prestressed Steel Lattice Girders."

Thursday, March 11th, 1954

Ordinary Meeting at 6 p.m., when Mr. S. J. Crispin, M.I.Struct.E., L.R.I.B.A., will give a paper on "Soil Stabilisation of Fine Materials."

Thursday, March 25th, 1954

Ordinary General Meeting, 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when Dr. P. B. Morice, B.Sc. and Mr. G. Little, M.Sc., will give a paper on "Load Distribution in Prestressed Concrete Bridge Systems."

Members wishing to bring guests to the Ordinary Meetings announced above are requested to apply to the Secretary for tickets of admission.

LONDON GRADUATES' AND STUDENTS' SECTION

The following meetings will be held at 11, Upper Belgrave Street, London, S.W.1.

Tuesday, January 12th, 1954

Mr. E. G. Robinson, A.M.I.Struct.E., on "The Design of Chimneys."

Tuesday, February 16th, 1954

Address by the President of the Institution.

Hon. Secretary : J. F. S. Pryke, B.A.(Hons.), Bushcroft, Slipe Lane, Wormley, Herts.

BRANCH NOTICES

LANCASHIRE AND CHESHIRE BRANCH

The following meetings have been arranged :—

Tuesday, January 12th, 1954

Joint Meeting with the Institute of Welding, Liverpool and District Branch, "The Plastic Theory and its application to Welded Structures," by a member of the staff of the British Constructional Steelwork Association. At the College of Technology, Liverpool, 7.15 p.m.

Wednesday, January 27th, 1954

Joint Meeting with the Institution of Civil Engineers, North-Western Association. Mr. Edgar Morton, M.Sc., on "Site Exploration and Foundation Problems."

Thursday, February 11th, 1954

Dr. F. G. Thomas, B.Sc., M.I.C.E., M.I.Struct.E., on "Composite Action in Structures."

Thursday, February 18th, 1954

Annual Dinner and Dance at the Grand Hotel, Manchester. The President and the Secretary will attend.

Wednesday, March 10th, 1954

Joint Meeting with the Liverpool Engineering Society. "The Uses of Aluminium for Structural Purposes," by a member of the staff of the Aluminium Development Association. At the Temple, 24, Dale Street, Liverpool, 6 p.m.

Thursday, March 25th, 1954

Joint Meeting with the Reinforced Concrete Association, North-Western Branch. Mr. G. P. Bridges, M.I.Struct.E., A.M.I.C.E., L.R.I.B.A., on "The Design and Construction of Reinforced Concrete Silos and Bunkers."

All meetings, unless otherwise stated, will be held in the Reynolds Hall, College of Technology, Manchester, at 6.30 p.m., preceded by tea at 5.45 p.m.

Joint Hon. Secretaries: A. S. Sinclair, A.M.I.Struct.E., 17, The Circuit, Cheadle Hulme, Cheshire; M. D. Woods, A.M.I.Struct.E., 58, Spring Gardens, Salford, 6.

MIDLAND COUNTIES BRANCH

The following meetings have been arranged :—

Friday, January 22nd, 1954

Mr. W. G. Carter, M.B.E., B.Sc., A.M.I.C.E., A.M.I.Struct.E., on "True Economies in Planning, Designing and Detailing of Structures."

Tuesday, February 9th, 1954

Mr. E. Shepley, B.Sc., A.M.I.C.E., on "Prestressed Concrete Framework for Liverpool University Medical School." At the Supper Room, The King's Hall (Queen Street Baths), Queen Street, Derby, 7 p.m.

Friday, February 26th, 1954

Mr. C. B. Brewington, B.Sc., A.M.I.C.E. (Graduate), and Mr. J. W. Fortey, A.M.I.Struct.E., A.C.T. (Birmingham), on "A Method of Structural Analysis by Large-Scale Models."

Friday, March 26th, 1954

Mr. H. V. Hill, M.Sc., A.M.I.C.E., A.M.I.Struct.E., on "The Load-Bearing Capacity of Metal Structures."

All meetings, except where otherwise stated, will be held in the James Watt Memorial Institute, Great Charles Street, Birmingham, at 6 p.m.

Hon. Secretary: L. A. Firminger, A.M.I.Struct.E., 656, Chester Road, Erdington, Birmingham, 23.

GRADUATES' AND STUDENTS' SECTION

Friday, January 29th, 1954

Mr. D. H. New, B.Sc.(Eng.), A.C.G.I., D.I.C., M.I.C.E., M.I.Struct.E., A.M.I.Mech.E. (Member of Council), on "Some Practical Applications of Prestressed Concrete," at the James Watt Memorial Institute, Great Charles Street, Birmingham, at 7 p.m.

Wednesday, March 31st, 1954

Address by the Chairman of the Midland Counties Branch, followed by the Annual General Meeting.

Hon. Secretary: H. L. Bramwell (Graduate), 139, Wood Lane, Handsworth Wood, Birmingham, 20.

NORTHERN COUNTIES BRANCH

The following meetings have been arranged :—

Wednesday, January 6th, 1954

Joint Meeting with the Northern Architectural Association, at Newcastle.

Wednesday, January 13th, 1954

Joint Meeting with the Institution of Civil Engineers. Mr. A. P. Clark on "Lackenby Works." At Middlesbrough at 6.15 p.m., preceded by tea at 5.45 p.m.

Tuesday, February 2nd, 1954

Mr. A. J. Harris, B.Sc.(Eng.), A.M.I.C.E., on "Prestressed Concrete in Civil Engineering Works." At Middlesbrough.

Wednesday, February 3rd, 1954

The above meeting will be repeated at Newcastle.

Tuesday, March 2nd, 1954

Professor W. Fisher Cassie, M.Sc., Ph.D., F.R.S.E., M.I.C.E., M.I.Struct.E., on "Pavement Structures," at Middlesbrough.

Wednesday, March 3rd, 1954

The above meeting will be repeated at Newcastle.

Thursday, March 18th, 1954

Ladies' Guest Night, at Middlesbrough.

Friday, March 19th, 1954

Ladies' Guest Night, at Newcastle.

All Meetings will commence at 6.30 p.m., the Middlesbrough Meetings being held at the Cleveland Scientific and Technical Institution, Corporation Road, and the Newcastle Meetings in the Neville Hall, near the Central Station.

Hon. Secretary: O. Lithgow, A.M.I.Struct.E., 4, Stoneleigh Avenue, Acklam, Middlesbrough.

NORTHERN IRELAND BRANCH

The following meetings have been arranged :—

Tuesday, January 12th, 1954

Details to be announced.

Tuesday, February 9th, 1954

Annual Dinner and Social Function, at the Grand Central Hotel, Belfast, 6 p.m. Visit of the President and the Secretary of the Institution.

Tuesday, March 2nd, 1954

Details to be announced.

Hon. Secretary: A. H. K. Roberts, B.A., B.A.I., M.I.C.E.I., M.I.Struct.E., "Barbizon," 26, Dunlambert Park, Belfast.

SCOTTISH BRANCH

The following meetings have been arranged :—

Monday, January 18th, 1954

At the Royal Technical College, Glasgow, 6.30 p.m. "Jetties and Fenders" by Professor A. L. L. Baker, B.Sc., M.I.C.E., (Vice President). Joint Meeting with the Institution of Civil Engineers, Glasgow and District Association. Visit of the President and the Secretary of the Institution.

Tuesday, January 19th, 1954

Annual Dinner and Dance at Grosvenor Restaurant, Gordon Street, Glasgow. The President and the Secretary of the Institution will be present.

Tuesday, February 9th, 1954

At the Ca'doro Restaurant, Glasgow, 6 p.m. Mr. C. M. Wilson, A.M.I.C.E., (Associate Member) on "The Reconstruction of Portobello Power Station."

Thursday, February 25th, 1954

At the Elite Hotel, Edinburgh, 6 p.m. Mr. Hugh B. Sutherland, S.M.(Harvard), A.M.I.C.E., F.G.S., A.M.I.Struct.E., on "Some Problems in Foundation Engineering."

Tuesday, March 16th, 1954

At the Ca'doro Restaurant, Glasgow, 6 p.m., Mr. F. R. Bullen, B.Sc., M.I.C.E. (Hon. Curator), M.I.Struct.E., on "Unusual Design for a Large Constructional Shop."

Hon. Secretary : G. Drysdale, A.M.I.Struct.E., "Niaroo," 33, Union Street, Motherwell, Lanarkshire.

SOUTH-WEST COUNTIES BRANCH

The following meetings have been arranged :—

Friday, January 15th, 1954

At Newton Abbot, 7 p.m. Mr. C. J. Woodrow (Graduate, Joint Hon. Secretary), on "The Tamar Bridge Proposal."

Friday, February 19th, 1954

At the Duke of Cornwall Hotel, Plymouth, 7 p.m. Mr. A. V. R. Hooker, A.M.I.C.E., M.I.Struct.E., on "Structural Engineering at Abbey Works, Margam."

Friday, March 19th, 1954

Film on "Bridging," at the Demonstration Theatre of the South-Western Gas Board, Union Street, Torquay, 7 p.m.

Joint Hon. Secretaries : E. W. Howells, M.I.Struct.E., 10-12, Market Street, Torquay, Devon ; and C. J. Woodrow, "Elstow," Hartley Park Villas, Tavistock Road, Plymouth, Devon.

WALES AND MONMOUTHSHIRE BRANCH

The following meetings have been arranged :—

Tuesday, January 26th, 1954

At Cardiff. Structural Quiz.

Wednesday, January 27th, 1954

At Swansea. Structural Quiz.

Wednesday, February 17th, 1954

At Swansea. Junior Members' Evening.

Tuesday, March 9th, 1954

At Cardiff. Joint Meeting with The Institution of Civil Engineers. "Barry Dry-Dock Reconstruction."

Friday, March 26th, 1954

At Swansea. Annual Dinner. The President and the Secretary of the Institution will be present.

Wednesday, March 31st, 1954

At Swansea. Mr. S. Woolf on "Recent Developments in Timber Structures."

Meetings at Cardiff will be held at the South Wales Institute of Engineers, Park Place, at 6.30 p.m.

Meetings at Swansea will be held at the Mackworth Hotel, at 6.30 p.m.

Hon. Secretary : G. R. Brueton, A.M.I.C.E., A.M.I.Struct.E., 86, The Exchange, Mount Stuart Square, Cardiff.

WESTERN COUNTIES BRANCH

The following meetings have been arranged :—

Friday, January 8th, 1954

Mr. A. J. Harris, B.Sc.(Eng.), A.M.I.C.E., on "Hangars at London Airport—Design of Large-Span Prestressed Concrete Beams."

Friday, February 5th, 1954

Mr. J. E. Collins, A.R.I.B.A., A.M.I.Struct.E., on "The Industrial Architect and Engineer."

Wednesday, February 17th, 1954

Annual Dinner at the Royal Hotel, Bristol.

Thursday, March 4th, 1954

Combined Meeting with the Institution of Civil Engineers. Details to be announced.

Unless otherwise stated, all meetings will be held in the University of Bristol Geology Lecture Theatre, at 6 p.m., preceded by tea at 5.30 p.m.

Hon. Secretary : E. Hughes, A.M.I.Struct.E., 23, Southdown Road, Westbury-on-Trym, Bristol, 9.

YORKSHIRE BRANCH

The following meetings have been arranged :—

Wednesday, January 20th, 1954

Mr. H. C. Husband, B.Eng., M.I.C.E., M.I.Mech.E., (Member of Council), and Mr. K. H. Best, B.Eng., A.M.I.C.E., A.M.I.Struct.E., on "Reconstruction of a Soaking Pit Building."

Wednesday, February 17th, 1954

Mr. H. D. Morgan, M.Sc., M.I.C.E., on "Driving and Testing of Piles."

Friday, March 12th, 1954

Annual Dinner and Dance, at Parkway Hotel, Bramhope, Leeds, 7 p.m.

Wednesday, March 17th, 1954

Mr. E. Lightfoot, M.Sc., B.Sc., A.M.I.C.E., A.M.I. Struct.E., on "Dynamic Stresses in Structures."

Friday, March 26th, 1954

Joint Meeting with the Yorkshire Association of the Institution of Civil Engineers, in Hull. Professor A. L. L. Baker, B.Sc., M.I.C.E., (Vice-President), on "Jetties and Fenders."

All meetings will be held at the Great Northern Hotel, Leeds, at 6.30 p.m., except where otherwise stated.

Hon. Secretary : E. Wrigley, A.M.I.Struct.E., 17, The Drive, Alwoodley, Leeds.

UNION OF SOUTH AFRICA BRANCH

Branch Hon. Secretary : A. E. Tait, B.Sc., A.M.I.C.E., A.M.I.Struct.E., P.O. Box 3306, Johannesburg. During week-days Mr. Tait can be contacted in the City Engineer's Department, Town Hall, Johannesburg. 'Phone: 34-1111, Ext. 257.

Natal Section Hon. Secretary : E. G. Bennett, A.M.I.Struct.E., c/o The Reinforcing Steel Co., Ltd., P.O. Box 49, Merebank, Durban.

Cape Section, Hon. Secretary : R. Stubbs, M.I.Struct.E. The Reinforcing Steel Co., Ltd., P.O. Box 2692, Cape Town.

Prestressed Steel Lattice Girders*

By R. A. Sefton Jenkins, B.Sc., A.C.G.I., A.M.I.C.E., A.M.I.Struct.E.

Synopsis

The prestressing of steel lattice girders has been done in the past by Professor Magnel and by F. J. Samuely. The method used by Professor Magnel is to keep straight or curved high tensile wires within the depth of the girder. The main advantage being that, for a given load high tensile wires are cheaper than mild steel. If the prestressing device is kept within the depth of the girder it is not possible to develop tension in either of the booms due to this prestressing. If the prestressing device is brought below the bottom chord (assuming the girder is to take vertical loading) then the bottom chord is put into compression and the top chord into tension by the prestressing.

Unlike prestressed concrete, where the elastic deformation due to the application of an external load produces practically no change of the load in the prestressing wires, with a steel beam the effect is considerable and is in fact a great advantage as the greater the load the greater the force in the prestressing bar.

The system has been used on a factory at Harlow New Town, where 60 ft. span girders with a largest individual member of a $1\frac{3}{4}$ in. × $1\frac{3}{4}$ in. × $\frac{1}{4}$ in. angle have been built.

Introduction

The first published account of the use of prestressed steel structures in this country was by Professor Magnel when he read a paper before this Institution in 1950. He showed that if the bottom chord of a lattice girder were prestressed with high tensile wires, economies were obtained.

Since then considerable work in this field has been carried out by F. J. Samuely. The latest example of his work is a garage in Wigan.

The main advantage described by Professor Magnel was that the cost to stress ratio for high tensile steel is lower than for mild steel.

As has been pointed out by Mr. Samuely, one of the objects of any prestressing should be to counteract the forces in a structure due to the applied loads and to make the forces in any member what you want them to be. If a lattice girder is prestressed so the prestressing force is on the neutral axis of the girder along its whole length, then the effect is merely to put both chords in compression. If, however, the prestressing force is made to deviate from the neutral axis, in addition to equal compression in the chords, a bending moment is induced in the girder. It is thus possible to make this moment act against the moments induced by the loads. If the prestressing device is curved then it can be made to produce the maximum effect where it is most wanted. In addition, the curve will reduce the effect of the shear forces as in prestressed concrete.

If the prestressing is kept within the depth of the lattice girder the effect of it is to induce compression only into both booms. However much the prestressing deviates from the centre line of the girder, it is not possible to induce tension, as the compression on the two booms will always be more than the effect of the moment. If, however, the prestressing is brought outside the depth of the girder, tension can be developed.

Donovan H. Lee and O. J. Masterman touched very briefly on this idea in the discussion arising out of Professor Magnel's paper.

Principles

The method of prestressing discussed in the remainder of the paper is confined entirely to cases where the prestressing device is below the bottom chord of a lattice. If this is done, a set of conditions occurs which is comparable with those obtained by prestressing a rectangular concrete beam ; for instance, it is well known that it is possible to prestress against the dead load with a concrete beam and a set of conditions as in Fig. 1 is obtained.

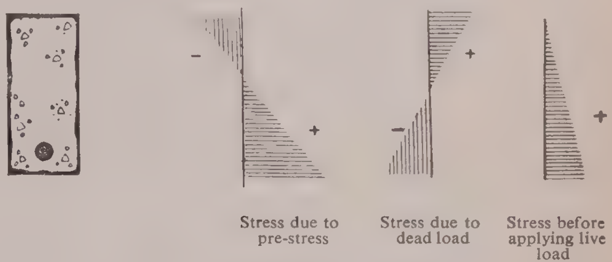

Fig. 1—Prestressed concrete

If a prestressing force is applied to a steel lattice girder so that the force is below the bottom chord of the girder, the same effect can be obtained (see Fig. 2).

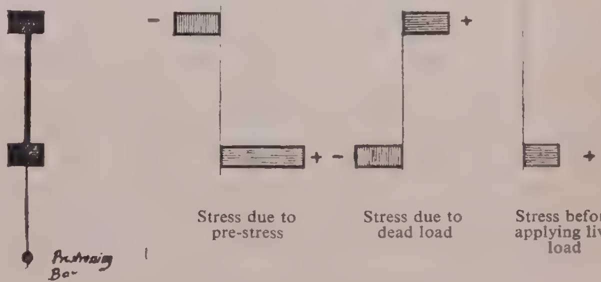

Fig. 2—Prestressed steel

The effect of the prestressing on the girder can be divided into firstly a moment tending to hog the girder, and secondly, a compression divided equally between the two chords of the girder.

One method of obtaining this effect is to hold the prestressing device away from the bottom chord in the manner of a queen post truss : (Fig. 3).

Fig. 3—Queen post girder

When a load is applied to a structure of this kind, whether the lower bar is prestressed or not, this load will increase the load in the lower bar. In this respect the prestressing of steelwork has advantages which are

*Paper to be read before the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1, on Thursday, February 25th, 1954, at 6 p.m.

not found in prestressed concrete, because very much smaller prestressing forces need be applied under no load or partial load conditions than are actually in the structure under full load conditions, and, also, as will be shown later, since the forces in the girder due to the prestressing are in opposition to those due to the external load, any increase in the one causes an increase in the other.

The effect of this, when the bottom bar is prestressed, is to increase the force in the prestressing bar as the load is increased.

So far, only one section along the length of the girder has been considered.

If the girder carries a uniformly distributed load along its length, the compression and tension in the top and bottom chords will vary parabolically along the length of the girder. It is desirable that the prestressing device should counteract this with a tension in the top chord and a compression in the bottom chord that also vary parabolically. It would be ideal if it were possible to cancel out completely both these forces. Unfortunately by this method it is not possible to do so completely. As has been mentioned above, the effect of the prestress is to induce in effect a compression and moment into the girder, so that there will always be more compression in the bottom than tension in the top.

Taking a queen post truss, the tension induced in the top and bottom chords will vary along the length as shown in Fig. 4.

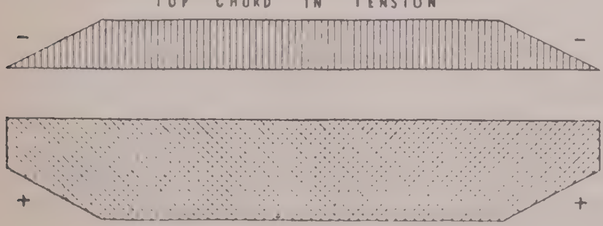

Fig. 4—Distribution of stress along length of a Queen post girder

The same variation along the length of the girder can be obtained by bending the girder, as in Fig. 5.

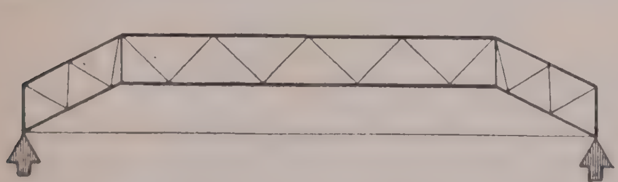

Fig. 5—Bent girder singly prestressed

A closer approximation to a parabolic variation along the length of girder can be obtained by superimposing two sets of prestressing (see Fig. 6).

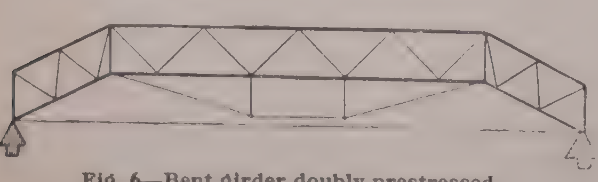

Fig. 6—Bent girder doubly prestressed

By varying the amount of prestress in each system a very close approximation to a parabola can be obtained.

Needless to say, the maximum stresses in the girder do not necessarily occur when the maximum load is spread along the full length of the beam, but, as in suspension bridges and other redundant structures, the maxima at different sections occur when a portion or portions are loaded.

Factory at Harlow, Essex

The factory will eventually have a floor area of 156,000 sq. ft. The first stage of 105,000 sq. ft. includes an area of 100 ft. × 210 ft. of concrete shell roof, flanked on either side by two areas of monitor light factory 80 ft. × 240 ft.

The monitor light portion consists of 60 ft. span prestressed steel beams at 24 ft. centres between which light secondary beams span, supporting either monitor frames or flat roofing.

The whole of the steelwork apart from the columns and a few items like gusset plates is fabricated from high tensile steel angles manufactured by re-rolling discarded railway lines. A number of tests on these angles have shown that both in compression and tension they are rather stronger than if they were made of steel to B.S. 548. However, for simplicity the stresses that are specified for steel to B.S. 548 in B.S. 449 are used in design. It has been found that for roof beams, trusses, etc., even with unprestressed lattice beams, the form of construction that has been developed shows economies over mild steel construction. When the beam is prestressed the savings are even greater, and, of course, the longer the span the greater the saving.

The method of fabrication of the high tensile lattice beams is interesting. The angles forming the beam are cut and the holes automatically punched. As the depth of various lattice beams has been standardised to four standard depths, the number of fixtures necessary is a minimum, especially as the depths are all multiples of each other. The parts are then assembled in a jig and the whole placed on a roller table. Each joint of the lattice then passes under a squeeze riveting machine which automatically squeezes the rivet and moves the whole on to the next joint. Small rivets up to ½ in. diameter are squeezed cold ; with larger sizes an additional step is introduced, just prior to squeezing the rivet is heated by means of two electrodes.

In the original design the girder was prestressed by two systems as shown in Fig. 8. The prestressing force in this scheme was obtained by tightening balloon cables with rigging screws. The cables had been produced during the war for balloon barrages and were extremely high tensile steel wires (145T/sq. in. ultimate) of 6/.072 in. construction and galvanised. In the final scheme 1¼ in. diameter bars of mild steel were used. As was mentioned earlier, the object of this double prestressing is to produce a bending moment diagram due to the prestress that resembles the bending moment diagram due to the applied forces as nearly as possible. Owing to the originality of the idea Harlow Development Corporation and the Steelwork Contractor decided to load test a pair of prototype girders. The prototype was tested (see Appendix B for details of this) and work was started on the production of the remainder. Harlow Development Corporation's client decided for various reasons completely unconnected with any structural aspect that they did not wish to have two systems of prestressing. The girder was hurriedly redesigned and the result is shown in Fig. 9. As will be seen, this resulted in a considerable increase in the amount of steel necessary in the girder, not only in the booms for bending but also in the diagonals for shear. In this particular case it is possible to prestress each girder before the application of any dead load. The stress in some of the members approaches the allowable limit.

Obviously if it had exceeded this limit the method would be to prestress the girders after the application of the whole or part of the dead load. The girders were initially sent to the site in three parts, the centre 40 ft. length and the two outer sloping portions. These were assembled and the prestressing bars attached. Later, complete 60 ft. lengths were riveted together in the factory, thus cutting out the need for fitted bolts at the corners. The bars had turnbuckles incorporated in them which were tightened until they were carrying the required tension. The tension was measured by measuring the strain with a strain gauge designed to fit

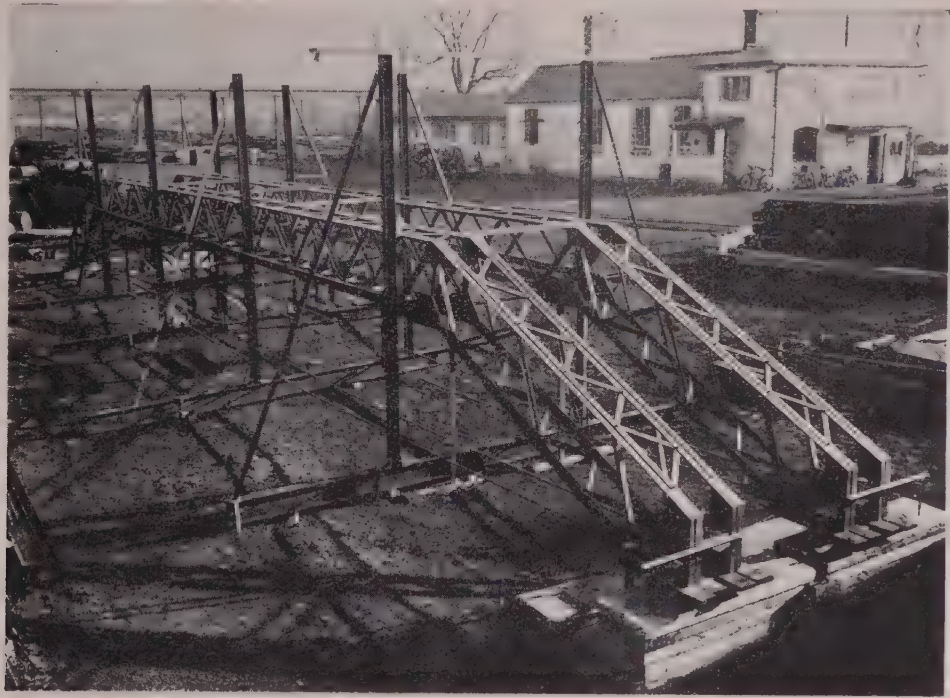

Fig. 7—Photograph of prototype beam

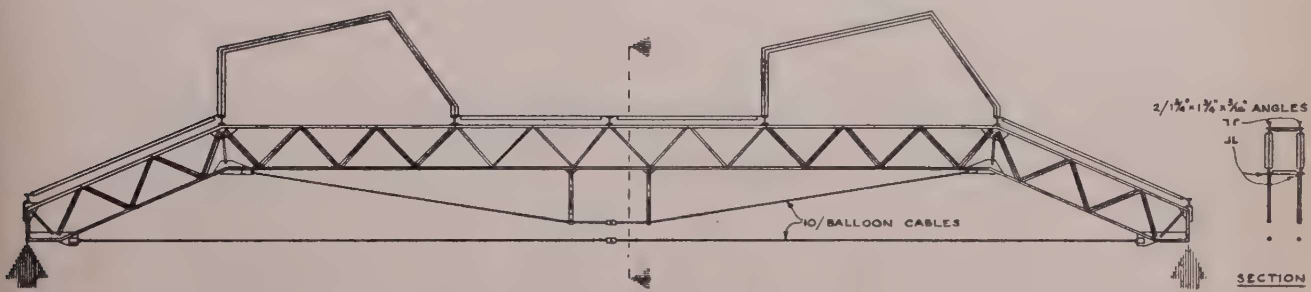

Fig. 8—Original design of beam

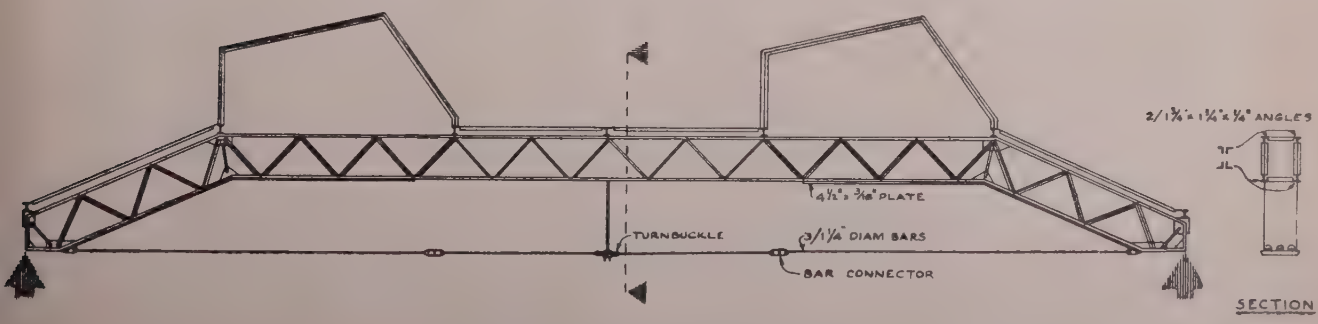

Fig. 9—Revised design of beam

into small holes drilled in the bar (see Fig. 11). The girders were hoisted into position and the work completed in the normal manner.

Conclusions

It may be thought by some that, where steelwork is prestressed within the depth of the girder, the economies that are produced on paper by the savings in steel are to a large extent offset by the extra work that is involved, especially as this is of an unusual character. It is, however, claimed by Professor Magnel that this is not so.

One of the difficulties in assessing the savings made by any new idea is the basis of comparison. It is usually difficult to produce, say, an unprestressed steel design to compare with a prestressed design which is equal in all respects, which is as carefully designed, and which fulfils all the same requirements exactly. In the case of the factory mentioned above the contract was awarded on the basis of competitive tendering for the steelwork as a whole, including design.

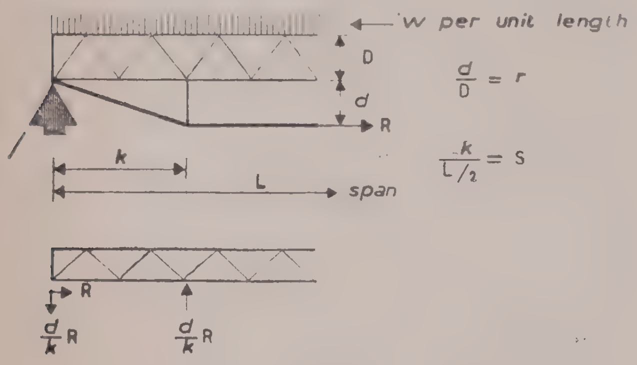

Fig. 10—Diagram for calculation

As far as the actual prestressing operation itself was concerned it was found that two junior engineers and a labourer could prestress eight girders in a morning without undue hurry.

It might be thought that the overall depth of the girder including the prestressing is greater. But taking the case of the Harlow factory the total depth is approximately 6 ft. for a 60 ft. span—which is the same depth as a conventional mild steel girder might be. However, the actual girder is only 2 ft. 2 in. deep instead of 6 ft., so that instead of having struts in the lattice of the order of 12 ft. long, they are only 4 ft. long, so that smaller and more economical members can be used.

Acknowledgements

The author would like to mention his indebtedness to his clients, Sommerfelds Ltd., and especially their directors, Mr. K. J. Sommerfeld and Mr. O. Schalscha, for their confidence in him in putting forward the design of the factory to Harlow Development Corporation, to whom the author is also indebted for accepting the design.

Mr. G. T. Goalen, of the Architects' Department of Harlow Development Corporation was exceptionally helpful in making the architects' point of view fit the engineers', whilst Mr. D. A. Cox, of the Engineers' Department, had the unenviable task of checking and made many helpful suggestions.

The strain gauge was loaned to the author by Mr. E. W. H. Gifford, Consulting Engineer, of Southampton.

Mr. F. J. Samuely is to be thanked for kindly reading the paper and making several helpful suggestions.

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APPENDIX A

Method of Calculation

The method used to solve the structures that have been encountered so far, has been with the use of strain energy. The structure shown in Fig. 6 is basically a structure with two redundancies. The prestressing bars are taken as the redundant members and the prestress in the bars is treated as an initial lack of fit of the bar. The girder can be analysed either as a beam with a bending moment or as a pin jointed structure with forces in each member. The answer is much the same in either case.

The first move is to find the differential of the strain energy of the whole structure with respect to each redundancy. Expressions in the following form are obtained :—

$$\frac{dU}{dR_1} = -aW + bR_1 + cR_2 = P_1$$

$$\frac{dU}{dR_2} = -dW + eR_1 + fR_2 = P_2$$

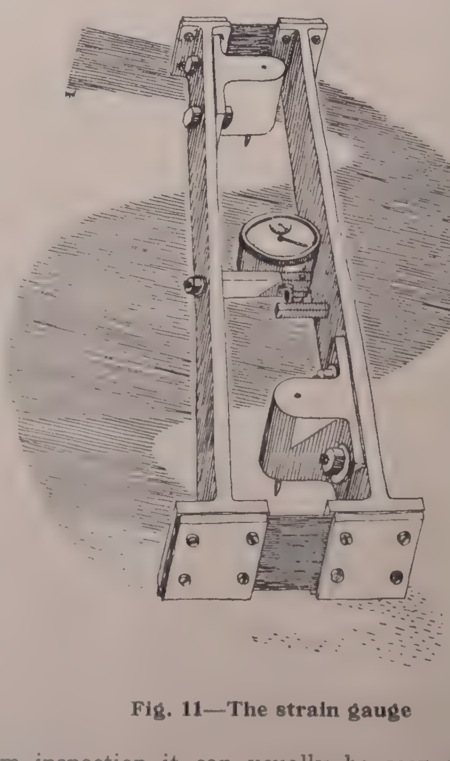

Fig. 11—The strain gauge

From inspection it can usually be seen what prestressing forces are required under maximum load. These are inserted together with appropriate value of W, giving values for P_1 and P_2. The latter will both be constant for all values of W.

It merely remains for the values of R_1 and R_2 to be calculated for various types of loading and the forces in the members checked to see that the structure is safe.

For instance if the beam is straight and the prestressing in the form of a queen post as below, an expression as follows can be derived.

$$\frac{dU}{dR} = \frac{DL}{12E_B I_B}\left\{RD\left\{3 - 6rs - 8r^2s + 12r + 12r^2\right\} - \frac{wL^2}{8}\left\{4 - 4rs^2 + rs^3 + 8r\right\}\right\}$$

$$+ \frac{R}{AcEc}\left\{L + \frac{2d}{R}\right\}$$

where E_B = Young's Modulus for the Beam.

E_C = Young's Modulus for the prestressing arrangement.

I_B = Moment of Inertia of the beam.

A_C = Area of the prestressing arrangement.

It should be noted that not only is the bending moment in the beam considerably reduced by prestressing but also that the shear up to the queen post strut is reduced.

APPENDIX B

A loading test was made on two full size beams of the original design for the Harlow factory. (See Fig. 8.) The balloon cables were replaced by 1 in. diameter rail steel bars. The beams were set up on blocks about 1 ft. clear of the ground and at 4 ft. centres as shown in Fig. 7. The four frames with vertical legs at the sides of the beams were to ensure that the whole arrangement did not move sideways under load.

R.S.J.s were placed between the two beams in positions corresponding to the position of secondary beams in the completed structure. Bundles of angles were then placed on the R.S.J.s so that the load corresponded approximately to the full live load plus dead load ($29\frac{1}{2}$ tons) in the completed structure.

The object of the test was to verify that the calculations and theoretical basis were borne out in practice. To this end, holes were drilled in top and bottom members so that a demountable strain gauge (see Fig. 11) could be applied before and after loading.

The theoretical load in each member was calculated as described earlier, taking a strain energy summation of each member, treating the member as pin jointed at each end. This was worked out for when the girder was loaded with full load and when unloaded (i.e., loaded only with its own self weight).

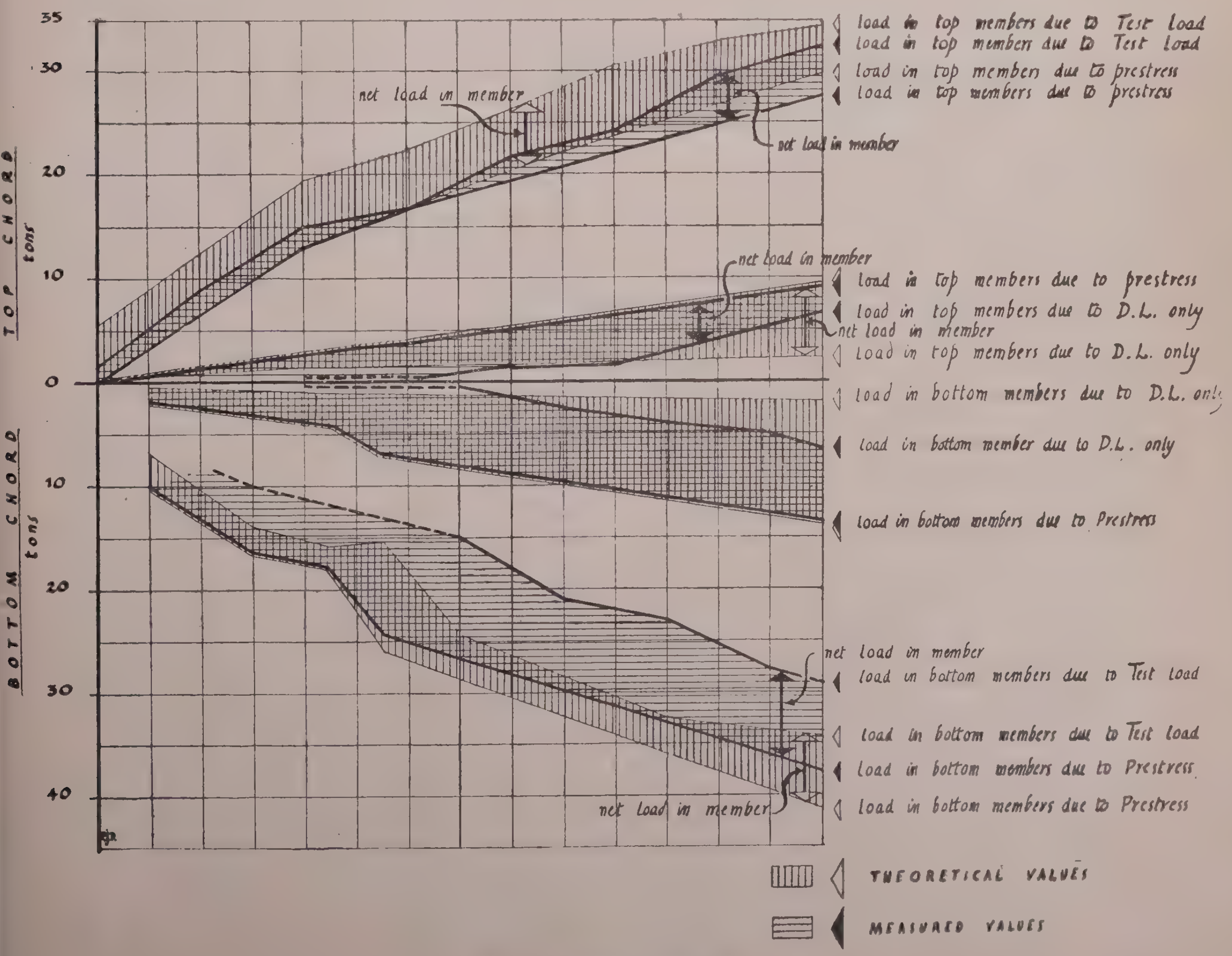

Fig. 12—Result of test load

Readings of the strain gauge to obtain completely unloaded condition were taken with the girders lying on their sides. The girders were prestressed by tightening the turnbuckles and measuring the load applied by means of the strain gauge. Measurements were taken of the horizontal contraction, and it was found that this, instead of being approximately 1 in. as calculated, was of the order of 3 in. Unfortunately the upward deflection of the girders before and after prestressing was not recorded accurately, as during the prestressing process the frame (see Fig. 7) used as a zero was accidentally knocked out of position and level, but the upwards camber of the girders was between $2\frac{1}{2}$ in. and 3 in. against a calculated camber of just over an inch. The reason for both of these exceeding the calculated values was almost certainly due to the fact that black bolts with 1/16 in. clearance holes were used at the junction of diagonals giving a small amount of movement at each joint which was taken up by prestressing. Later beams were riveted throughout and the calculated values were found to agree in practice to within $\frac{1}{8}$th inch.

Under load the ends of girder move apart. The actual expansion ($\frac{3}{8}$-inch) on the test girder was 1/32 inch less than the calculated movement. The vertical deflection of the centre point under the test load was $1\frac{1}{4}$ in. against a theoretical deflection of $1\frac{3}{4}$ in. The loads in the members under full load obtained from the strain gauge readings have been plotted, using as a basis the theoretical effect of the measured load in the prestressing members. These can be compared with the theoretical values (Fig. 12). A similar set are shown for the girder under its own load.

It will be seen from Fig. 12 that the agreement between the theoretical variation of load in the prestressing bars and the measured variation is good. The agreement between the theoretical and measured net load in members is not in every case quite so good but the variations are not large.

One probable reason for this variation is that the calculations are based on a completely pin-jointed structure, whereas the top and bottom boom were continuous throughout. Similarly, it must be remembered that the " loads " in the members are derived from strain gauge readings which will include in the reading any effects of local bending, etc. To observe fully this effect it would have been necessary to take far more readings in various positions on each member than were taken in practice.

In view of the foregoing it is felt that the test showed that the method of calculation is fully justified.

Book Reviews

The Marseilles Block, by Le Corbusier. Translated by Geoffrey Sainsbury. (London : The Harvill Press, 1953.), 72 pp., 11 in. × $8\frac{1}{2}$ in., price 21s.

Le Corbusier's Marseilles Block is well-known, but the history and background which led up to this unusual solution of a housing problem is described and illustrated with considerable humour by the architect in this volume.

The design has been in no way controlled or limited by any building regulations, and the author traces the history of homes and home-life back to the most primitive forms, and then sets out to design a " unit of habitation " based on fundamental principles. The result will not suit everyone.

A flat, or rather a cross between a flat and a maisonette, which is the basis of each unit in this block, which is some 75 ft. from back to front and only 11 ft. wide, and having windows only in the extreme ends, is decidedly unusual. When grouped as one of a mass of some three hundred and sixty flats, it does, however, result in a very large housing scheme being concentrated in one block, thus allowing Garden City-like precincts to be provided in a comparatively small area of land and at the same time avoiding the blight of scattered development.

The photographs in this book are excellent. The sketches are amongst the roughest which have ever been reproduced, but despite this they do convey what is in the author's mind, and that is saying a lot when one learns that the theme incorporated in this design was not produced as a result of concentrated effort at the time that Le Corbusier was commissioned for this particular job in 1945, but is the physical result of continuous study and development from as long ago as 1907.

From an engineering standpoint, the idea of raising this immense block on legs clear of the ground in order that there should be a free passage of air and unobstructed views around and through the building, is well worthy of attention.

A. H. L.

Studies in Elastic Structures, by A. J. S. Pippard. (London : Edward Arnold & Co. 1952.) $9\frac{1}{2}$ in. × $6\frac{1}{4}$ in. 361 pp. 60s.

This book is largely based on an extensive library of papers on advanced structural analysis which Professor Pippard and his associates have produced in the last two or three decades. Each chapter deals with a separate study and is complete in itself, with the terminal references evidence of the background of careful, but adventurous, research from which the work emerges.

In some ways these studies may be considered a development from " The Analysis of Engineering Structures," the familiar textbook by Professors A. J. Pippard and J. F. Baker. For example, the chapters on the elastic arch rib, the bow girder and the masonry arch are covered to a lesser extent in this earlier work. Among the remaining studies those on inter-connected bridge girders, multiple lattice frames, cable bracing, stresses in a restrained pipe line and open panel structures will also be of interest to the structural engineer.

Classical methods of analysis are used throughout and formulæ are often evolved in the face of rather tedious mathematics. The Castigliano theorems are plainly preferred to the Virtual Work equations, and no use is found for the Column Analogy. The treatment follows a pattern of resolute application of a restricted number of analytical methods, with continuous equivalent systems devised to replace limited finite conditions where they are intractable. Confirmatory experiments usually follow.

The volume is admittedly of first interest to the structural specialist, who should find it both a model and inspiration. The practising structural engineer will gain from it an insight into the way problems are engaged in the research field ; often an exploratory work to shape many of the recommendations required in the various design codes.

R. H. E.

The Effect of Plastic Yield in Bending on Mild Steel Plate Girders*

By A. Lazard, Chef de la Division des Ouvrages d'Art de la Societe Nationale des Chemins de Fer Francais

I. Theory

I want to speak to you about plastic yield in bending of plate web beams in mild steel.

The elementary theory is well-known and is, in most cases, realised in practice, and my object is to verify agreement or divergence between theory and fact.

The theory supposes

1. An ideally plastic material with a well-defined and extensive range of plasticity.

2. A redistribution of stresses in the section which is most strained such that all stresses become equal to the yield stress (positive and negative) : the bending moment becomes equal to the theoretical full plastic moment.

In statically determinate structures plastic failure occurs when this happens.

3. In redundant structures a redistribution of bending moments such that in the first section which is most strained, the bending moment remains equal to the theoretical full plastic moment whilst the bending moment rises in the second most strained section and is eventually equal to the theoretical full plastic moment : plastic failure then occurs with two plastic hinges at these two sections.

4. The predicted results are not affected by residual stresses such as produced for example by a change in level of a support.

II. Statically Determinate Beams (progressive increase of loading)

(1) Plain Joists

Let us now see how conditions change in the course of an experiment. We will start by considering statically

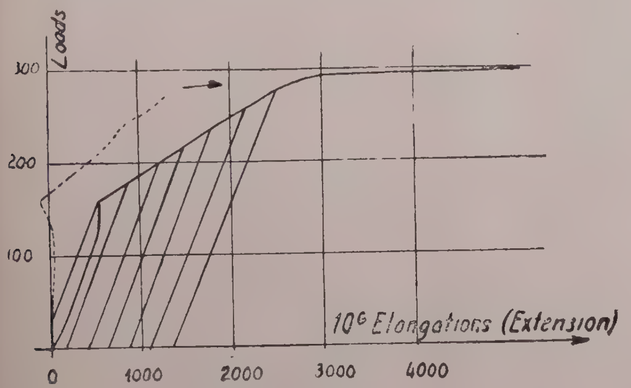

Fig. 1

determinate beams : for example a beam supported in two points and loaded in two points so that the middle of the beam is subjected to a constant bending moment.

(a) First, let us consider joists (plain joists).

Plastic yield proceeds by sudden jumps from one point to another before a whole space is made plastic, so that plastic yield develops later on more equally and isotropically.

The beginning of plastic yield in one given point depends on the residual stresses existing there, due to "working" the steel and to the manufacture of the beam and to previous treatment of that beam. When active bending stress plus residual stress are equal to the yield stress plastic yield occurs ; that is permanent

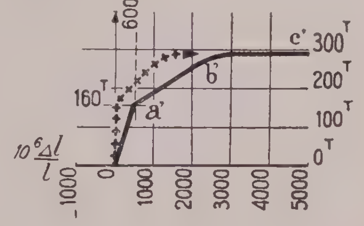

Fig. 2

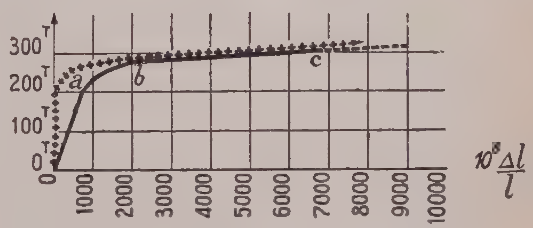

Fig. 3

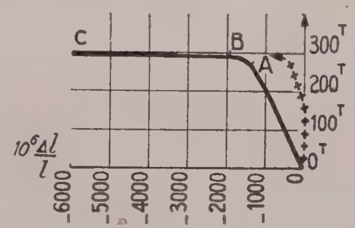

Fig. 4

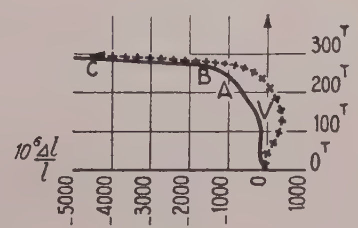

Fig. 5

deformations appear after the load has been removed. Residual stresses vary from one point to another so that the beginning of plastic yield is heterogeneous in the beam.

This may be observed visually or by extensometers usually used in structural engineering. It might be thought that with more accurate extensometers, such as microscopic scales, these phenomena would seem to be

*Read before a Joint Meeting of the Iron and Steel Institute, The British Section, Societe des Ingenieurs Civils de France and the Institution of Structural Engineers, at 4, Grosvenor Gardens, S.W.1, on Wednesday, April 15th, 1953.

homogeneous. This, however, is not the case and metallurgists studying plastic yield of crystals in the molecular range have arrived at the same conclusion as ourselves, namely, that plastic yield is discontinuous.

(b) Considering now diagrams of : extension in relation to bending moment it is seen that the results are peculiar. Figs. 1, 2, 3, 4, 5 and 6 are obtained from an experiment on 1 meter high H beams, that is the largest rolled

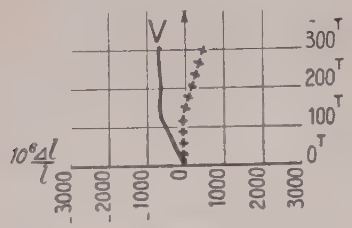

Fig. 6

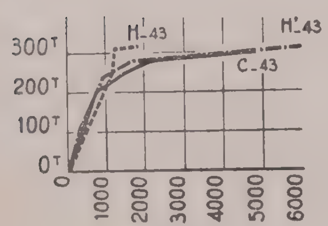

Fig. 7

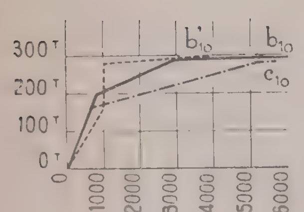

Fig. 8

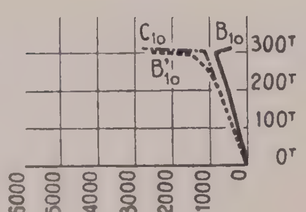

Fig. 9

sections in the world. I should like to draw your attention especially to the tension zone with the three stages. I am not able to explain the first stage of plastic yield, since this is not envisaged in the elementary theory, about which I am quite sure. (Figs. 1 and 2.)

(c) I placed a number of extensometers at various points—in this case, they were Baldwin or Phillips electrical strain gauges—one close to the other.

You can see the results in Figs. 7, 8 and 9, and that when plastic yield (generally a large extension) takes place in one point, the extensions in the neighbourhood of the points vary, sometimes considerably, becoming even, in some cases, a shortening. The beginning of plastic yield in one point modifies the whole structure around it.

(d) If you consider now the deformation of a transverse section, you will see (Figs. 10 and 11) that the Bernoulli hypothesis gradually becomes untenable. The Navier hypothesis which follows it must also become inapplicable and we may well expect the plastic law of redistribution of stresses to develop.

(e) If we consider the Piobert-Luders-Hartmann lines (see Fig. 12), we can see two different phenomena. In the tension zone of the beam we find the well-known net of two types of straight lines intersecting at an angle of nearly $\frac{\pi}{2}$ both on the flanges and on the web. In the compressed part we find a net of parallel straight lines, perpendicular to the axis of the beam, both on flanges and web. If we try to explain this by the theory of Professor Baes, we find that in both cases (tension and compression zones) there exists a compression stress acting on the surfaces parallel to the axis. In the tension zone the Mohr circle intersects the shear stress axis and we have shear lines. In the compression zone the principal Mohr circle, with one principal stress equal to 0, is tangential to the shear stress axis and we have shear wrinkles.

This compression stress may, in some way not as yet understood, modify the distribution of the normal stresses in a section. If we accept some such hypothesis, we can expect that this distribution will be influenced in the following way (Fig. 13) : in the compression zone

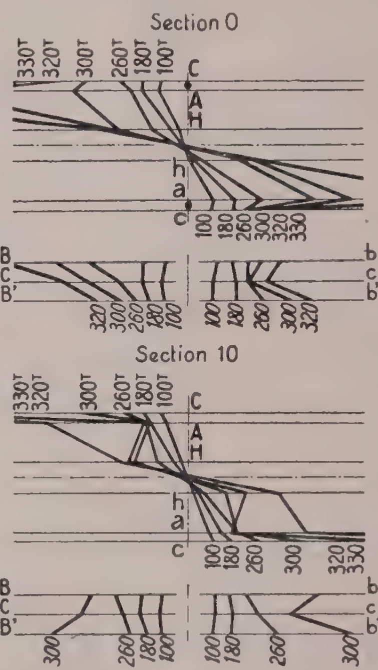

Fig. 10

increase in the stresses ; in the tension zone decrease in the stresses ; finally, however, this influence is probably of secondary importance and can usually be ignored.

(f) In our own experiments it seems that the entrance into the plastic range begins generally in the tension flange. What, however, happens in the compression flange ? If no care is taken to prevent buckling, failure generally occurs by buckling or twisting or lateral deflection in the compression zone. Sometimes simultaneous buckling of the flange and of the web is observed. In other cases, twisting of the whole section or lateral deflection of the compression flange occurs. In general, then, with large joists it is impossible to reach the plastic failure as theoretically described : all we can say is that there is plastic buckling or plastic twisting, or plastic instability : but the bending moment (failure moment) is very near to the ultimate when care has been taken.

On the other hand, with small joists, which are often tested in laboratories, plastic failure occurs as theoretically described.

(g) A very important question then arises. Is the actual ultimate bending moment equal to the theoretical value ? Generally it is ten or fifteen per cent. higher, sometimes even more.

What is the explanation ? As the part taken by the web is theoretically small it is usual to calculate the

theoretical full plastic moment with the yield point of the flanges. In fact the yield stress of the metal in the web is higher than that of the flanges : the difference is generally 3 or 4 kg/mm², that is about 15 per cent. more, and finally the part taken by the web seems to be higher.

This occurs when a constant bending moment is applied. With one load only we may expect that the bending moment at plastic failure would be a little

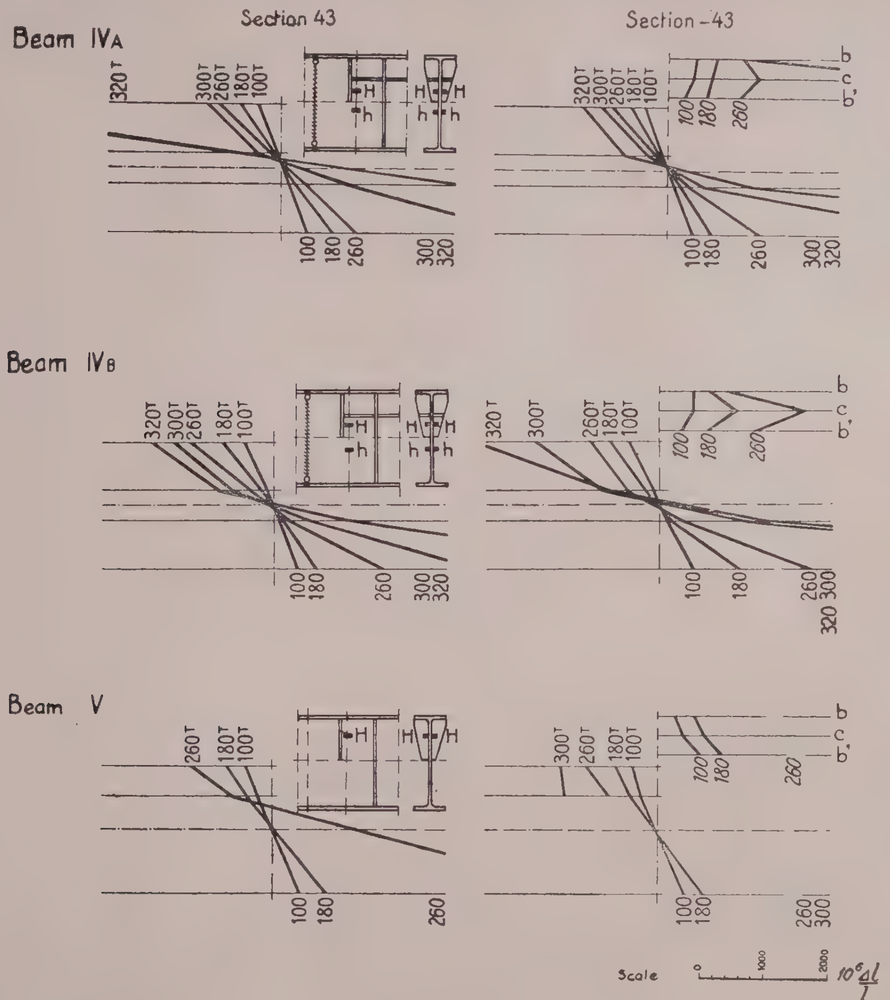

Fig. 11

Fig. 12

higher, say two or three per cent. more, because plastic failure occurs only when a sufficient volume around the location of load has been made fully plastic.

(2) Joists Jointed by Butt Welds (progressive increase of loading)

A number of our experiments on 1-meter high H beams were actually carried out on two joints jointed by a vertical butt weld. We tried some different welds, and some different ways of welding. The welds in the flange were of types V or X ; the welds in the web were generally of type X. We welded in many cases with the beam always in the same position (especially with the welding process tried by the French Railways called "C.G.H.," which proved to be satisfactory), in other cases with the beam rotating in two big circles. The butt welds were continuous or interrupted by holes of different shapes or sizes.

We found no difference between all these various types and the results were similar to those of plain joists. I shall accordingly not make any difference between plain joists and joists jointed by butt welds.

(3) Holes in Flanges (progressive increase of loading)

What happens now when holes are present in the web and especially in the flanges ? The answer is different, depending on whether the holes are drilled or punched.

(a) If drilled it seems that the holes do not change appreciably the result, that is, the plástic failure occurs in nearly the same manner and for the same, or approximately the same, bending moment. This is at variance with theory since holes must diminish the area subjected to stresses equal to the yield stress. In other words, drilled holes maintain nearly the capacity for plastic deformation if they do not exist.

(b) With punched holes, collapse occurs with a bending moment which is almost the same as before but, and this is a characteristic feature, there is no plastic failure

but a sudden break or rupture starting from the hole. It is in fact similar to what is often called brittle fracture. When observing gauges or dials, we cannot anticipate rupture, and in this case the beam is perhaps a little stiffer. Punched holes accordingly seem considerably to diminish the capacity of the beam to deform so that rupture occurs, but even so we cannot say if that is due to the transformation of the metal caused by punching (crystals are greyer and not so clear), or to the injury caused to the surface surrounding the hole. As yet I have not seen experiments with punched holes which have been subsequently reamed, but it is expected that results will be the same as for drilled holes. If it is the case, that will prove brittle fracture with punched holes is due to the surface injury.

(4) CYCLES OF LOADING (joists)

(a) As yet I have described experiments in which the bending moment was progressively increased (even when the load was being released). An important question

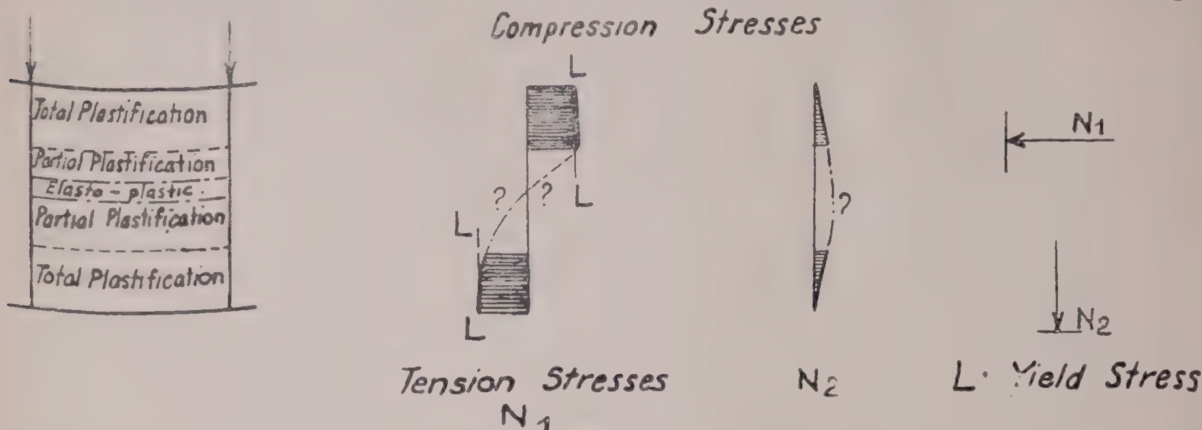

Fig. 13

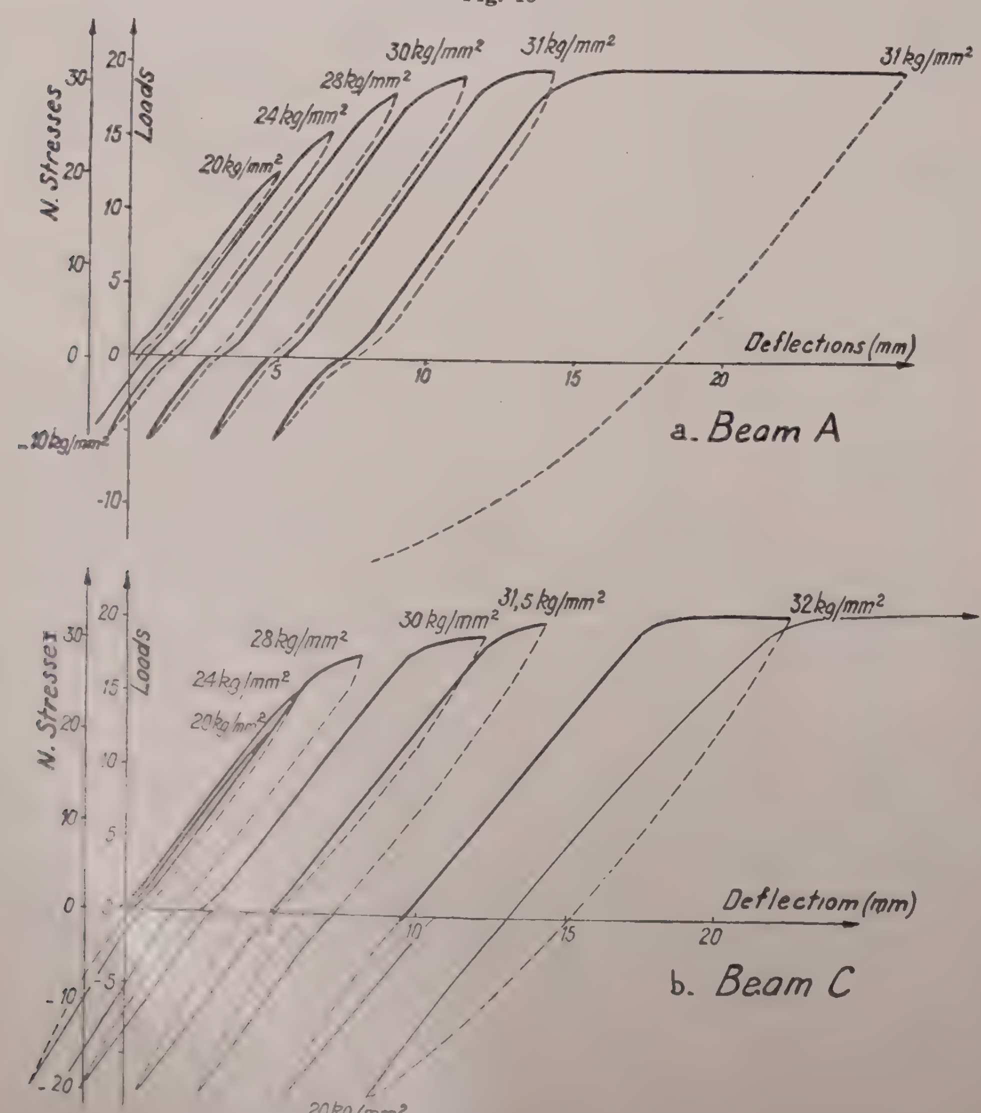

Fig. 14

was to find out the behaviour under varying moments such as :

fluctuating moments, that is moving between two positive limits ($+A$ to $+B$),

repeated moments, that is moving between one limit and zero (O to $+B$),

alternating moments, that is moving between one negative limit and one positive limit (higher in absolute value) ($-A$ to $+B$),

(b) We soon saw that for plain joists no difference occurred when the load was progressively increased, and we can say that plastic failure occurs with practically the same bending moments and that if there are differences they will be small.

(c) For alternating and oscillating moments the diagrams showing deflections in relation to moments show the Bauschinger effect, and in case of decrease, there is a straight line till the moment changes sign, when

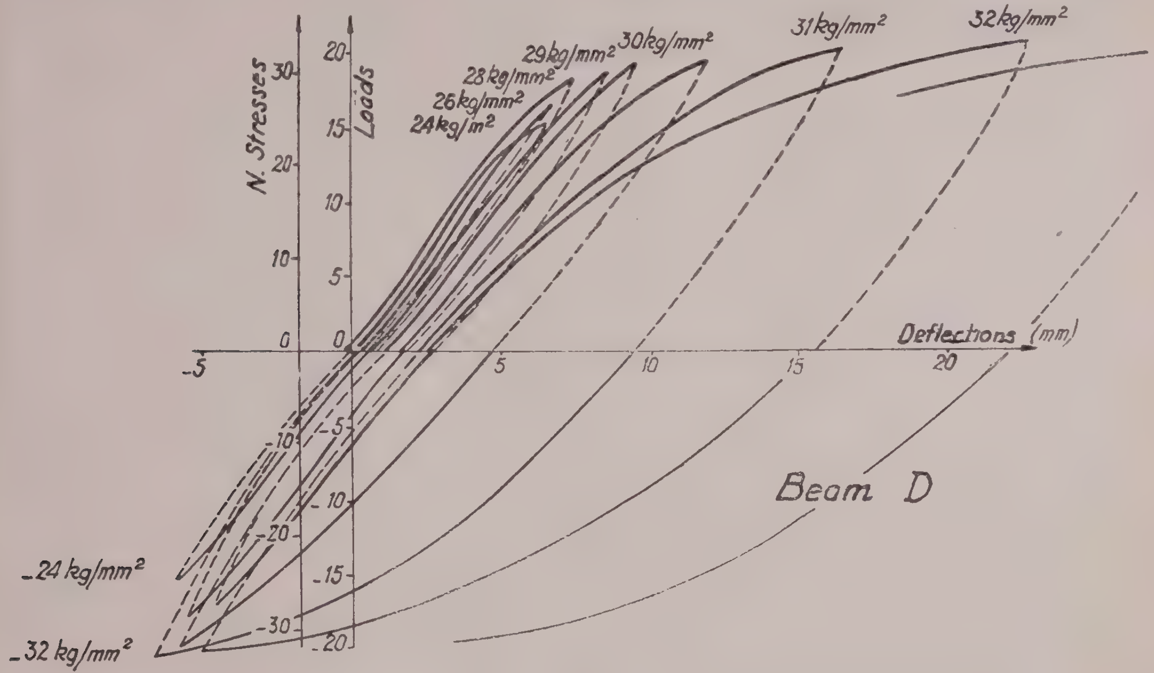

Fig. 15

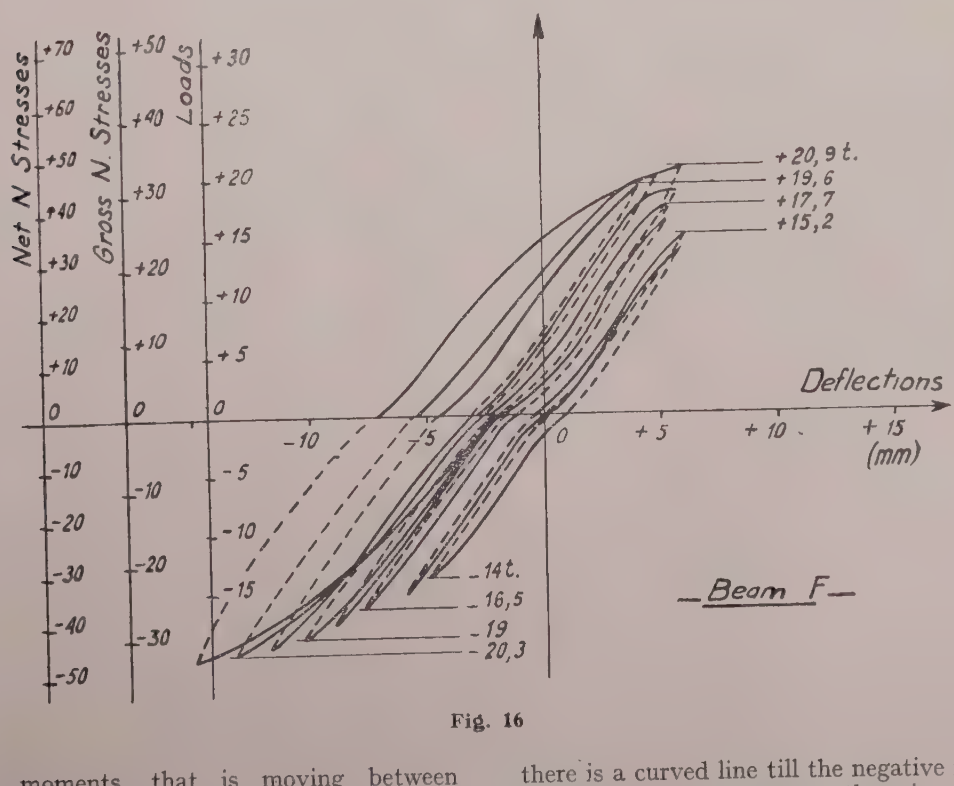

Fig. 16

oscillating moments, that is moving between + and — the same limit ($-B$ to $+B$).

We carried out these experiments, but owing to the capacity of the laboratory machinery, we were obliged to experiment only on *IPN* 200 joists.

there is a curved line till the negative moment is maximum. In case of decrease, there is a straight line till the moment becomes equal to zero and then there is a new curved line. The hysteresis loop is well shaped and there are two permanent deflections : one, when going

from positive to negative moments, the other in the opposite direction, when going from negative to positive moments. (Figs. 14 and 15).

(d) When a new cycle is established the hysteresis loop moves and develops, but no rule seems to be laid down (contrary to the assertion made after some earlier tests).

(5) Cycles of Loading (holes in flanges)

(a) After such an illuminating experiment, we carried out the same with drilled and punched holes. With drilled holes the conclusion is that plastic failure occurs at almost the same bending moment with progressive increase of loading—the difference is higher than with joists without holes, but remains small. Nevertheless, small cracks originate in the holes, and perhaps if we have subjected our beams to a greater number of similar cycles (in this case 20), we might have reached the point of rupture. The diagrams of deflection in relation to moment are very much the same as before (Fig. 16).

(b) With punched holes, the diagrams of deflection in relation to moment are different. The hysteresis loop is very narrow and rupture suddenly occurs, cracks originating from inside the holes, the starting point being the minute injuries to the skin. The ultimate bending moment is somewhat less than when there are no holes. (Fig. 17.)

(6) Some Conclusions for Plain and Perforated Joists

(a) It is convenient to introduce here the notion of what I call "Navier stress"; that is the quotient of the actual bending moment by the modulus of resistance ($\frac{I}{v}$ or Z) of the section.

$$N \text{ stresses} = \frac{M}{Z}$$

In the elastic range the Navier stress is equal to the actual bending stress; in the plastic range the Navier stress is fictitious and gives an indication of the bending moment.

When there are no holes we have to consider one Navier stress only, that one calculated with the modulus of resistance of the gross section, and this has to be compared with the yield stress.

When there are holes we have to consider, on the other hand, two Navier stresses: one in the gross section, the other in the net section and the two—which for brevity we refer to as net N. stress and gross N. stress—must be compared both with the yield stress (L) and to the ultimate strength (R) of the metal in the flanges.

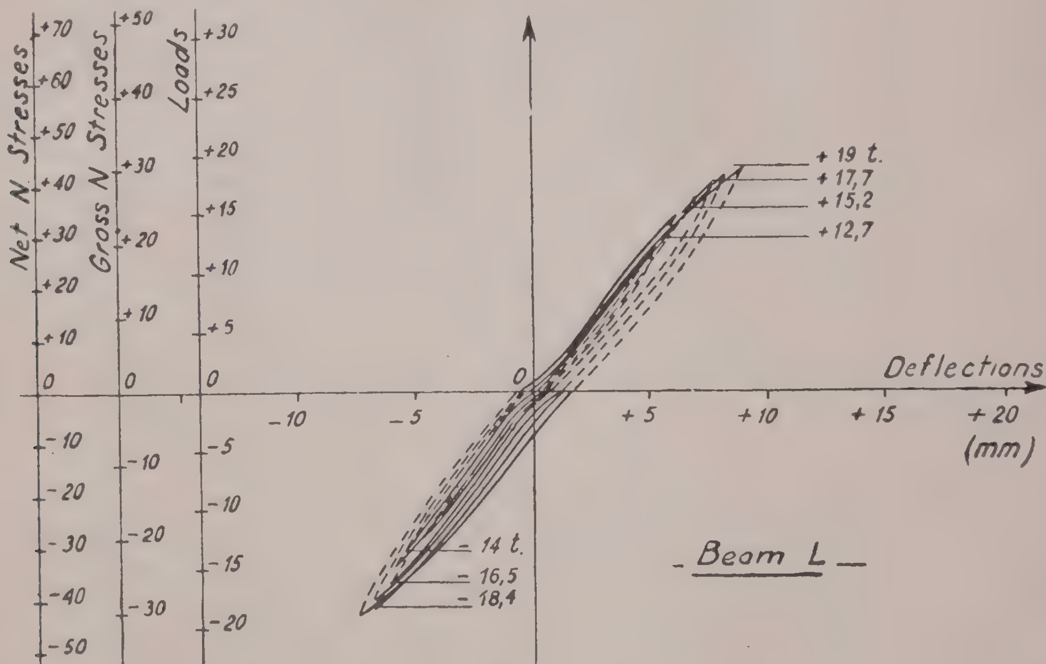

Fig. 17

The experiments about which we have information are:

1. By the French Steel Structure Contractors' Association, and

2. By the French Steel Structure Contractors' Association and the French Railways.

They were both carried out on rolled joists *IPN* 200 with 17 mm. diameter holes. So the ratio of moduli of resistance, and subsequently that of gross and net N. stresses is equal to 1.43. On the other hand the ratio $\frac{R}{L}$ is also close to 1.43. In the conclusions which follow it is accordingly impossible to say whether the phenomena are dependent on L alone (as in the case of plain joists), or on R alone (as expected for punched holes at least) or on both L and R. This question, which seems important for the use of other steels or other materials, should be solved by new tests on joists or other sizes.

Nevertheless, the results are as follows:

With drilled holes

1. The gross N. stress is *less than* R (about 0.9 R for progressive increase of loading; 0.8 R for cycles of loading),
and *always greater than* L (about 1.3 L for progressive increase; 1.15 L for cycles).

2. The net N. stress is *always greater than* R (1.3 R progressive ; 1.1 R cycles) and much greater than L (more than 1.6 L progressive ; 1.6 L cycles).

With punched holes

1. The gross N. stress is *less than* R (about 0.8 R progressive ; 0.7 R cycles) and *very close to* L, but slightly higher.

2. The net N. stress is *greater than* R in progressive loading (1.4 or 1.8 R) *and very close to* R but slightly higher in cycle ; and always much greater than L (1.4 or 1.8 L progressive ; 1.5 L cycles).

(b) A very important question is to know what can be done with a plain joist which has failed, or nearly so, as a result of plastic yield. We tested such joists and we found that they were able to support large loads. That is, after attaining a permanent deflection, they behave like a new elastic joist till loads reaching nearly the ultimate plastic bending moment. These loads are much greater than the permissible loads generally allowed by the ordinary regulations.

These phenomena are quite clear for a joist subjected to plastic failure as a result of progressive increase in loading. They are a little more complex for joists loaded to destruction by cycles because they seem to depend, in a way as yet not properly understood, on the time spent between the first failure and the next loading. Rest seems to enable the elastic qualities of the beam to recover.

You see at once the great importance of these results[1], which indicate that plain joists recuperate after having been subjected to plastic yield, assuming it is possible to accept the permanent deflection obtained.

It is even possible to conceive processes in which permanent deflections are especially given so that joists behave elastically (strains and deflections strictly proportional to loads and moments) under live loads. It is in fact the extension to rolled joists of old processes applied to other metals or shapes and especially to bars or cables.

The processes may be mixed with pre-stressing. A Belgian company is actually developing on a large scale such a process with the " Preflex beam " in which pre-stressing of a concrete flange is obtained by elastic deformation of a joist.

(c) It may be asked what happens to the qualities of the metal strained in such a way as just described ? To our knowledge, it has never been proved that with plain joists subjected to plastic failure by progressive increase (even with release of load) that yield stress or ultimate strength have been changed.

On the contrary, our experiments on plain joists and on joists with holes subjected to loading cycles have proved a large increase in yield stress. This increase is such, that in some cases the new yield stress in the flanges becomes greater than the new yield stress in the web. In some cases ultimate strength becomes also higher. In all cases, elongation at rupture has a marked decrease.

(7) Welded Beams

Some experiments have been made with welded beams. If the flange and web materials have the same yield stress (but in fact this was very seldom the case ; generally the yield stress of the web is higher than that of the flanges), plastic failure occurs as theoretically described and the ultimate bending moment is scarcely higher than predicted.

(8) Riveted Beams

Very few experiments have been made with riveted beams. It seems that rupture must occur, but we have not sufficient information about the way in which holes are drilled or punched.

(9) Residual Stresses—Asymmetrical Sections

Important experiments have been made in Russia by Patton and Gorbunov to test asymmetrical sections and residual stresses introduced in welds, under repeated cycles of loading.

All these tests agree very well with the elementary theory and constitute one of the best examples to demonstrate it.

(10) Conclusions

We are now able to state conclusions for statically determinate beams.

(a) With plain joists and welded beams plastic failure occurs as theoretically described when precautions have been taken against buckling or twisting or lateral deflection. There is no difference between progressive increase of loading and fluctuating, repeated, alternating, or oscillating loadings ; in any of these cases, the ultimate bending moment is generally higher than predicted. The difference may be as great as 40 per cent., and in a number of cases much higher than the difference between the theoretical full plastic moment and the full elastic moment (this difference varying from 15 to 18 per cent. according to the joists). This has a great practical importance, since we can hope that the ultimate bending moment will be generally 30 per cent. higher than the full elastic moment.

Putting it in another way, we can say that the ultimate gross Navier stress is generally 30 per cent. higher than the yield stress. If the joist is of mild steel with a yield stress of about 24 kg/mm^2, we can expect an ultimate gross Navier stress of 32 or 33 kg/mm^2.

I shall deal later with permissible stresses (see Chapter VI).

(b) For beams with drilled holes it seems that the ultimate bending moment is very nearly equal to that of beams without holes. In my opinion, permissible loads should be only slightly less than for plain joists.

With punched holes, the position does not appear quite clear. From our experiments it seems that the ultimate bending moment is but a little less than with plain joists, but this is not necessarily true for any size of joists or beams. For other sizes of beams it may very well happen that the ultimate bending moment is a certain ratio of the ultimate strength of the material. On the other hand, brittle failure occurs with punched holes so that, in my opinion, permissible loads must be less.

III. Redundant Beams

(1) Progressive Increase of Loading

(a) Now we will deal with redundant structures, although I, myself, have not carried out experiments with them. I will only deal with the experiments of others, and these have been generally made with small beams. From a certain point of view, we must consider separately continuous beams and built-in beams on the one hand and portals on the other. Experiments on portals are a speciality of this country. You are,

[1]Similar results have been obtained by Horne in Cambridge on continuous beams with a very small square section and very few repetitions.

therefore, more competent in fact than I am, so I will not go into details.

(b) Nevertheless, I think we may consider two main facts.

1. The full plastic bending moment can only develop at the second strained point if there is a sufficient volume to make plastic or if the values of the two elastic bending moments at the two most strained points are not too close to one another, or, perhaps, if shear stresses are not too high.

2. When the yield stress is reached in a point a permanent elbow will be produced in the beam. Thus, when plastic failure occurs we have two permanent elbows. This is very different from the isostatic case where we have permanent deformations, but no elbows. That may be a great inconvenience in structural engineering, but in other respects it is a way of indicating that plastic yield has commenced.

Of course we may also have buckling, twisting, and in general all forms of instability. It will be assumed that dimensioning is such that no instability will occur or that the member has been suitably strutted or stiffened.

(c) With continuous beams it seems that the two bending moments tend really towards equality, as predicted by the elementary theory, but that cannot in fact happen and when equality is nearly reached a further large deformation occurs at the first strained point, which enters into the reinforcing stage (cold hardening) of the metal, deformations at the second point no longer increase. I think that is perhaps an effect of restraint due to the supports. (Fig. 18.)

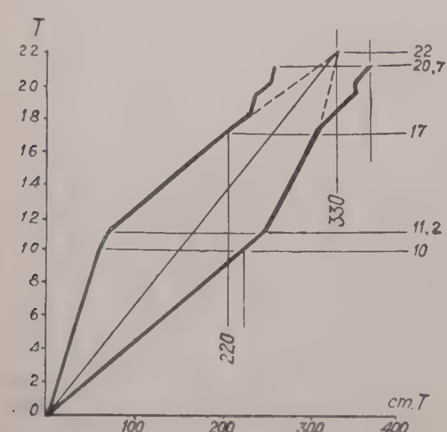

Fig. 18

(d) On the other hand, in portals where plastic hinges form generally under the points of loading and in the knees, and where there is no effect of restraint except where the two hinges are too near to one another (lack of plastifying volume or two practically equal elastic moments as stated in paragraph b1) (Fig. 19, Hendry's test), the two moments become in fact equal, and sometimes this happens twice or even more during loading. (Fig. 20.)

(e) Except when conditions described in paragraph (b) 1 are realised, the elementary theory of making two bending moments equal to the full plastic moment leads to an ultimate load twice that predicted when considering only theoretical full plastic moment in the most elastically strained section. In the case described in paragraph (b) 1, we may have only one and a half times the theoretical load. I think that the well-known Stussi experiment on a continuous beam with ends built in differently occurs within this category and that is perhaps why the ultimate load varies between the value

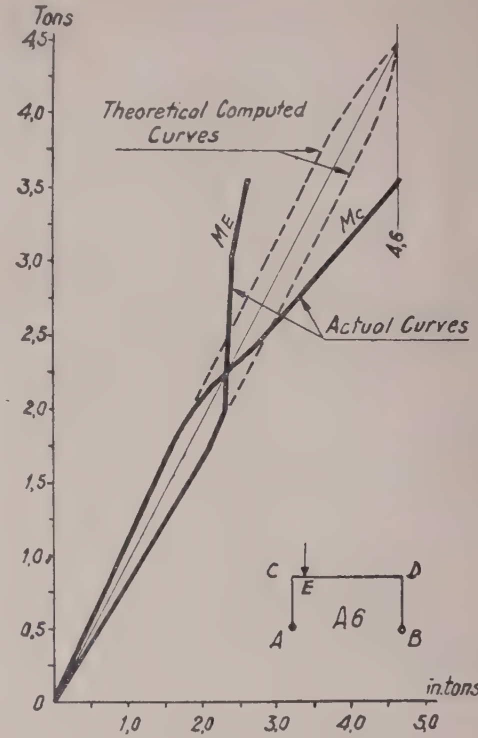

Fig. 19

of one for plastic moment, with an isostatic beam to two with a completely encastre beam.

(2) Moving Loads

(a) All this concerns the progressive increase of loads. There are practically no experiments with fluctuating, repeated, alternating and oscillating loadings. There

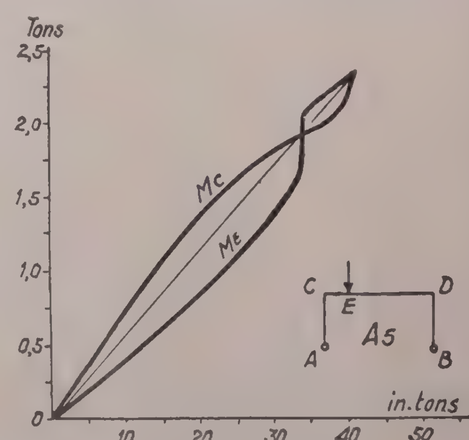

Fig. 20

are only two experiments published on a subject in which bridge builders are most interested : that is the effect of moving loads on a continuous beam. The subject was treated theoretically by Bleich's son in 1932 : his well-known theorem was corrected a little by Dutheil in 1948. But in my opinion the theorem is false and the analysis is more complex. I will not start a discussion here, but I will give the results of the experiments.

(b) The first one, by Klöppel in 1935, with an I joist 120 in two spans 1.50 m. each, with one load in the middle of each, one being constant, the other fluctuating between zero and a maximum. Bleich's theory pre-

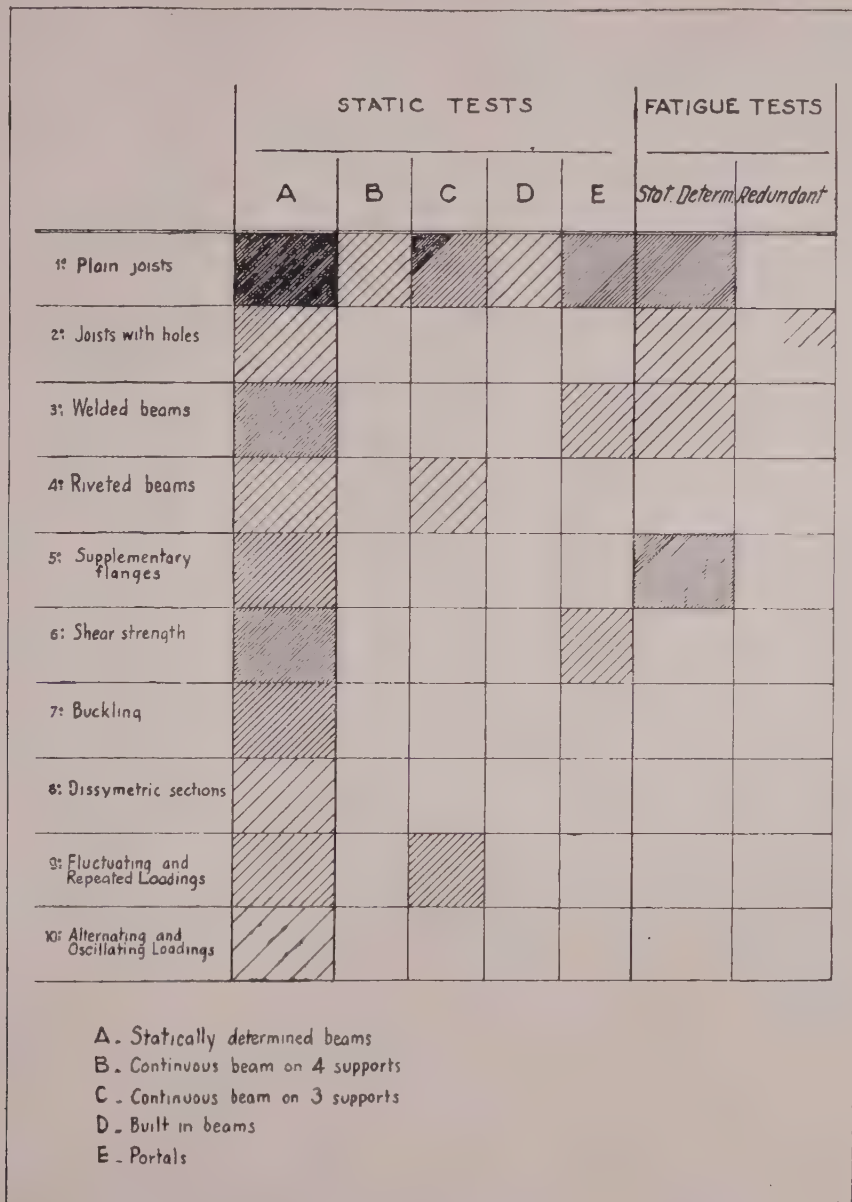

Fig. 21

dicted failure with a load of 4.2 t. After more than one million repetitions of a load of 5.83 t. the beam behaved very well.

(c) The second experiment, by the French Steel Structure Contractors' Association, intended to prove Dutheil's improvement, is more complex. There were two spans 1.30 m. each ; loads were located at $\frac{3}{10}$ points, from the central support. A square, a rhomb on the edge, and two joists were tested. Only one of the joists was a normal product, the other, the square and the rhomb were annealed. One load remained constant, the other varied between 1 t and a maximum (P). Five cycles were operated and the maximum (P) then was increased. With only five cycles of each loading, it was not easy to determine when deformations increase infinitely : so that conclusions are not definite and it is always possible to argue about what is to be considered as an ultimate load.

Nevertheless, as the predicted values were (in French tons)

	Square	Rhomb	1st Joist	2nd Joist
Bleich's theorem	12.2	8.6	11.7	11.2
Dutheil's theorem	14.6	12.1	12.8	12.3
The actual values were in Laboratory conclusions.	18	17	11.5	11.0
The Author's own conclusions.	17	15	10	10

You can see that for the square and the rhomb, that is for sections having a large volume for plastic yield, that is those having a large form factor, actual values are much larger than predicted.

On the other hand, with the two joists which have a small form factor, actual values are equal, in the best combination (Laboratory computation = Bleich's theory), or less than predicted. This confirms my theoretical own conclusions. There is still much to do on that problem.

(d) Professor Massonnet, of Liege, recently carried out new experiments on this subject. They are not yet published.

IV. Fatigue Tests

Coming now to fatigue tests. When in our experiments we tested cycles of fluctuating, repeated, alternating and oscillating moments, it must be understood that there was a limited number of cycles imposed, and at an ordinary rate. It is quite different when a beam is subjected to fatigue tests, that is one million times or more of a cycle of forces at great speed.

The interest of such tests has often been in dispute : there are mainly two schools of thought : one asserts that fatigue phenomena may not appear in ordinary structures, the other asserts the opposite. It is certain that in some cases fatigue phenomena can take place : for example in beams supporting motor turbines. Except in these cases, it is not easy to arrive at a conclusion. As for bridges, the French think that under ordinary conditions they do not take place, in spite of some cracks appearing in stringers that have been considered in other countries as the proof of those effects. But to have a good idea of the margin of safety, even under ordinary conditions, it might be interesting to know the behaviour of structures under fatigue tests.

We can find only very few experiments of that kind. The largest series was recently made in U.S.A. at Columbia University under the direction of Professor Wilson and the management of the Welding Research Council ; in this case specially welded beams have been tested. The results are complicated and the only simple conclusion I can emphasise is this : transverse welds as well as non-continuous longitudinal welds are much worse than continuous longitudinal welds ; in the same way incomplete supplementary flanges (part length) are worse than continuous flanges (full length).

V. Tests Still to be Carried Out

To conclude with a general view of experiments of bent beams I have prepared the following table where you can see from the density of black the number of tests made all over the world. (Fig. 21.)

Where you see much white it means that no experiment has been attempted in that case. A great deal of tests are still necessary to provide an answer to the question.

I will not say that it is impossible now to draw up schemes and build structures on the existing basis of design. The theory called in your country " Plastic Design " has made a good start and seems—with some reservations which I have emphasised in my paper—to agree very well with the facts as far as plain joists and welded beams are concerned. But we must be careful when holes appear in the most strained portion of a structure.

VI. French Practical Rules

I will now deal with permissible stresses and let you know what we are doing in this respect in France.

(a) You know that, in France, we have two sorts of regulations. The oldest ones for bridges and generally for works within the jurisdiction of the Ministry of Public Works ; the latest for houses and factories, namely for works within the jurisdiction of the Ministry of Reconstruction.

(b) Nothing has been done now to alter the regulations relating to bridges, but I just start to ask for some improvements at least for railway bridges.

(c) You perhaps know the regulations referred to as C.M. 46, issued by the Ministry of Reconstruction. We are endeavouring to improve them in numerous ways. One of these is the permissible stresses in bending for beams with constant section and plain web. I have proposed higher stresses than those finally adopted.

Some members of the Commission raise as an objection the magnitude of the deflection. In my opinion, these

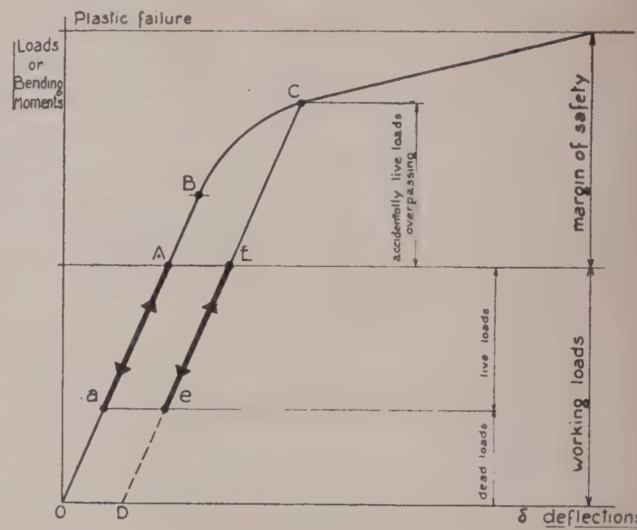

Fig. 22

criteria should not be mixed ; each one gives a limit and we must finally take the lowest.

We already have practical rules to follow for redundant structures. The further step consists in decreasing arbitrarily the modulus of resistance, Z, of the beam or joist, so that finally permissible stresses rise (both for statically determinate and redundant structures).

(d) The new clause referred to as 3.37 will state that the modulus of resistance Z may be increased arbitrarily by 15 per cent. when there are no holes or when there are drilled holes or punched holes subsequently reamed ; 8 per cent. when there are punched holes. When there are holes, the modulus of resistance to be considered is that of the net section.

This clause will apply only to I beams with full web having a constant section. It will therefore not be applicable to I joist reinforced by non-continuous flanges, this being a reference to the Wilson tests.

At my request, it has also been added, that the buckling of flanges and web shall be prevented as necessary by special constructive arrangements (struts, stiffeners, etc.).

When Article 3.37 comes into effect, the normal permissible stress being 14.4 kg/mm² for a material, the yield stress of which is supposed to be 24 kg/mm² (but not guaranteed), we can say for a beam statically determinate actual permissible stresses will rise to :

1. $14.4 \times 1.15 = 1.66$ kg/mm² for the gross section in plain section,
for the net section with drilled holes.

2. $14.4 \times 1.08 = 15.5$ kg/mm² for the net section with punched holes.

(e) It is not easy to give examples of redundant structures owing to the complexity of the rules laid down for them in Dutheil's theory. Nevertheless a clause specifies that these regulations are given as an example but that all other proved theory of plasticity may be used, with a general restriction : in any case the yield stress must not be exceeded, when calculations are made by the ordinary theories of Elasticity or Strength of Materials. I shall certainly apply those clauses in a specific case when tests are well known and when the differences between fact and the elementary theory of plastic design are known.

(f) For the isostatic cases, the majority of which have been investigated, I regret that a greater step was not taken in specific structures where the margin of safety must be well defined. In my opinion, advantage must be given first to plain sections, and secondly to drilled holes where failure occurs by plastic failure, increase of permanent deformation giving an indication of progress towards plastic failure.

Papers on the same Subject by the Lecturer

LAZARD, A. (1) Memoires de l'A.I.P.C. (I.A.B.S.E.). Volume 10, p. 101.
(2) Travaux : Mai 1950.
(3) ,, Novembre & Décembre 1951.
(4) 4° Congrès de l'A.I.P.C. (Fourth Congress of I.A.B.S.E.).
Preliminary Publication (p. 123).
(5) Final Publication (p. 125).
(3) and (4) give references to papers of other authors all over the world.

DISCUSSION

Introductory Remarks

Mr. J. E. SWINDLEHURST (President, British Section of the Société des Ingénieurs Civils de France ; Past-President, Institution of Structural Engineers), introducing the Lecturer, said M. Lazard was a pupil of the Ecole Polytechnique from 1928-1930 and entered the service of Ponts et Chaussées in November, 1933. As an Area Engineer he was responsible for all National and Departmental Road Services in the Department of the Moselle up to the declaration of war, and in 1940 was engaged in a similar capacity on National Roads and the Canal Service in the Department of the Loire.

During the war, from 1941-42, he was in charge of the scheme for motor roads in the neighbourhood of St. Etienne, and in 1943-1944 transferred from that service to join the Technical Service dealing with large-scale barrages, being engaged on the Barrage de Bort (Corréze) which involved studies of hydraulics, railroad deviation, soil mechanics and ancillary roadworks.

He became Engineer-in-Chief of the Ponts et Chaussées in 1950, and in 1951 was appointed Technical Counsellor at the Scientific Centre and Building Technique.

Throughout his career he had given particular study to matters relating to hydraulics, stability of earth embankments, the cracking of concrete and the plasticity of mild steel.

M. LAZARD then presented his paper, at the conclusion of which the Chairman proposed a vote of thanks. This was seconded by Mr. A. E. Chattin, Assistant Secretary of the Iron and Steel Institute.

Mr. J. S. TERRINGTON (The British Iron and Steel Research Association), introducing the discussion, wished to offer his own congratulations to the author both on the preparation and the presentation of the paper. He felt that the paper was particularly valuable because it not only gave the results of M. Lazard's own tests but also presented a comprehensive survey of the subject of plastic yield in simply supported beams, continuous beams and portal frames. M. Lazard had brought out the difference in the effects of continuous loading and cycles of repeated loading and had provided evidence of the characteristic way in which plasticity develops. Whereas, he said, we have for a long time been familiar with the use and properties of cold-drawn and twisted steel in reinforced concrete and more recently with drawn high-tensile steel for prestressed concrete, M. Lazard had now brought out the advantages of treating steel joists in a similar manner. The fact that the ultimate plastic bending moment can be as much as 30 per cent. higher than the full elastic bending moment was especially significant as well as the author's conclusion that drilled holes do not appreciably reduce the ultimate resistance. A timely warning of the possibility of brittle fracture developing at punched holes had been emphasised in the paper although the risk of this had been minimised by the use of drilled holes in modern methods of fabrication. In view of the increasing use of welding it was reassuring to hear the tests had confirmed that welding does not reduce the ultimate strength of the beam.

Mr. Foulkes' recent paper had shown that the strength of portal frames is relatively greater than for trussed members provided they are prevented from buckling and now M. Lazard's paper had shown what allowance, if any, should be made for holes in a section. In this connection it was interesting to see that the section should have a large form factor because that also controlled the resistance to lateral buckling which Mr. Terrington had to consider recently.

Mr. Terrington asked the reason for the conclusion that the strength of plain joists was related to the yield stress but that the strength of joists with holes was related to the ultimate stress. With regard to the idea for subjecting joists to plastic yield to increase their ultimate strength he felt that the necessity of applying a permanent deflection would not be well received by architects and he also felt that the reduction in ductility or percentage elongation must be a disadvantage and amount to a loss in factor of safety. Although the tests were carried out within the plastic range, Mr. Terrington emphasised that the actual use of the beam would still be within the elastic range and that this should not be lost sight of. Mr. Terrington also recalled that there were forms of construction such as crane gantry girders which carry heavy dynamic loads and which could not suffer risk of excessive temporary, still less permanent, deflection, and that the suggested higher working stresses should not be applied to types of construction such as these. In conclusion, Mr. Terrington again voiced his appreciation of M. Lazard's presentation of the paper and the meticulous way in which the tests had been carried out.

Mr. E. M. LEWIS spoke as one whose concern it was to apply in everyday engineering practice the knowledge gained from such theoretical and experimental work as M. Lazard had described, in order to give the best overall economic result.

He said the author had given considerable attention to the plastic flexure of beams containing holes, presumably with reference, in practice, to riveted or bolted constructions. Perhaps this was to some extent due to the tragic need for rebuilding so much old work in France during the past few years.

Continuing, Mr. Lewis gave his personal view—and in this he felt many of his friends and colleagues in Britain would concur—that fundamentally fabrication by welding is the right economic answer and that the decline of riveting and bolting will be greatly accelerated when somewhat artificial economic practices are abandoned. Thus, his main interest lay in that part of the paper dealing with welded beams. However, there might in the future be some economic reasons for plastically designed welded steel frameworks erected with friction grip bolted joints—that is to say joints of the type which have been developed in the United States of America particularly to overcome fatigue effects in bridge work where high-tensile steel bolts in oversize holes are highly prestressed to give a strong friction grip joint. He would be very interested to learn if M. Lazard had considered this possibility and, if so, whether any experimental work regarding the static load-bearing capacity—as distinct from the mainly dynamic U.S.A. tests—had been carried out or contemplated in France with a view to their incorporation in structures designed on the plastic theory.

The work which had been done on beams containing holes did, however, underline in Mr. Lewis's mind a point which had been worrying him for some time, that is, the temptation to practising engineers like himself to take for granted the fundamental properties of their structural steels. It would appear that the apparently anomalous high strength of beams containing drilled holes, the rather frightening brittle fractures of beams containing punched holes, the Stüssi effect, and so on, might all be linked with the work hardening properties of the steel. Thus, the value to the practising engineer of the most interesting results given by the author would be greatly enhanced if they could be correlated with the actual steels used. Could the author give any of the usual mechanical test data on the steels in the test beams so that they might be compared with typical British steel, and also had he any data on the strain hardening properties, the notch impact properties, and further, of great interest in relation to the punched hole brittleness, the low temperature notch brittleness? Information regarding chemical analysis, steel making and rolling techniques, might also be relevant.

Mr. Lewis then reiterated his view that in building construction particularly the right economic answer is the completely welded steel framework—what might be termed a "monometallic" structure—and he showed some slides of general interest of the British Welding Research Association's fatigue laboratory at Cambridge showing what he believed to be the first practical application in a building in Great Britain of Professor Baker's and the B.W.R.A.'s work on the plastic theory. The slides also served to illustrate how the problem of preventing plastic or elasto-plastic instability in such frameworks can be overcome at no extra cost by utilising the stabilising effect of the cladding, and to indicate the inherent stability of the completely "monometallic" structure.

Mr. F. A. Partridge said that M. Lazard's paper contained much information and many statements that would seem to upset some of the ideas held in this country on the behaviour of beams in the plastic range.

One very startling revelation was that holes in web and flanges of joists, loaded to collapse, behave as if they were not there, which would almost suggest that joints in beams might be made just as effectively by riveting as by welding.

Paragraphs (f) and (g) of Part II, 1, seemed so full of apparent contradictions and misstatements that Mr. Partridge had come to the conclusion that there were serious differences in our understanding of the Plastic Theory. He agreed with M. Lazard when he said in effect, in paragraph (f), that in both large and small joists, provided means were taken to prevent premature collapse through instability and buckling, then plastic collapse would occur as the theory predicted. But at the beginning of paragraph (g) he had said "generally it (the actual ultimate bending moment) is ten to fifteen per cent. higher (than the theoretical value), sometimes higher." Mr. Partridge thought that M. Lazard must apply the term "theoretical value" to the ultimate *elastic* moment of resistance. This was equal to $f_y Z$, where f_y was the yield stress and Z the elastic modulus of the section.

The ultimate *plastic* moment of resistance, however, was given by $f_y S$, where S was the *plastic* modulus of section, and for joists S was about 15 per cent. higher than Z. Was this the 15 per cent. that M. Lazard had in mind?

Then again, further on in paragraph (g) M. Lazard said that "the part taken by the web is relatively small." In fact, said Mr. Partridge, even at ultimate *elastic* moment of resistance the part taken by the web was in the neighbourhood of 20 per cent., while at ultimate *plastic* moment of resistance it was as much as 30 per cent. Those percentages were by no means small.

Continuing, Mr. Partridge said that according to our understanding of the Plastic Theory, in the collapse condition yield stress spreads to the neutral axis of the joist, and the ultimate plastic moment of resistance calculated on this basis had been confirmed very closely indeed by experiments at Cambridge University—in other words, the theoretical value agreed with the actual.

If, as M. Lazard suggested, the yield point for the material of the web could be as much as 15 per cent. higher than that of the flanges then it was true that the ultimate plastic moment of resistance of the whole section would be about $3\frac{1}{2}$ per cent. higher.

At the end of paragraph (g), M. Lazard said "Plastic failure occurs only when a sufficient volume around the location of load has been made fully plastic," and suggested that actual bending moment at failure is a little higher in the case of point loading as compared with a condition of loading producing constant bending moment. He attributed this to the fact that in the case of point loading a smaller volume of metal develops plasticity. Mr. Partridge said that our understanding was that the collapse condition had been reached when full plasticity had developed at a section whatever the loading, and in that condition any attempt to increase the load resulted merely in increased deflection. With constant bending moment, deflection prior to failure would, of course, be relatively large and the actual failure point somewhat indefinite. Consequently collapse would seem to occur earlier—perhaps this was what M. Lazard meant.

Fig. 1 showed the extent of plastic zones in beams of H section and of rectangular section, subject to point loading and uniform loading.

Mr. Partridge, referring to "Navier stress" mentioned in Part II, 6, said this was a conception which seemed to him completely unreal and full of pitfalls. The modulus Z was a geometrical property of the section linked directly with straight-line distribution of elastic

stress. It was the quantity which, divided into bending moment gives the elastic stress in the extreme fibres of the flanges of the section carrying the bending moment. For a beam bent beyond the elastic range the straight-line distribution of the stress across the section no longer applied, the material being in an elasto-plastic condition. When the section became fully plastic, that is, when yield stress had spread from the flanges to the neutral

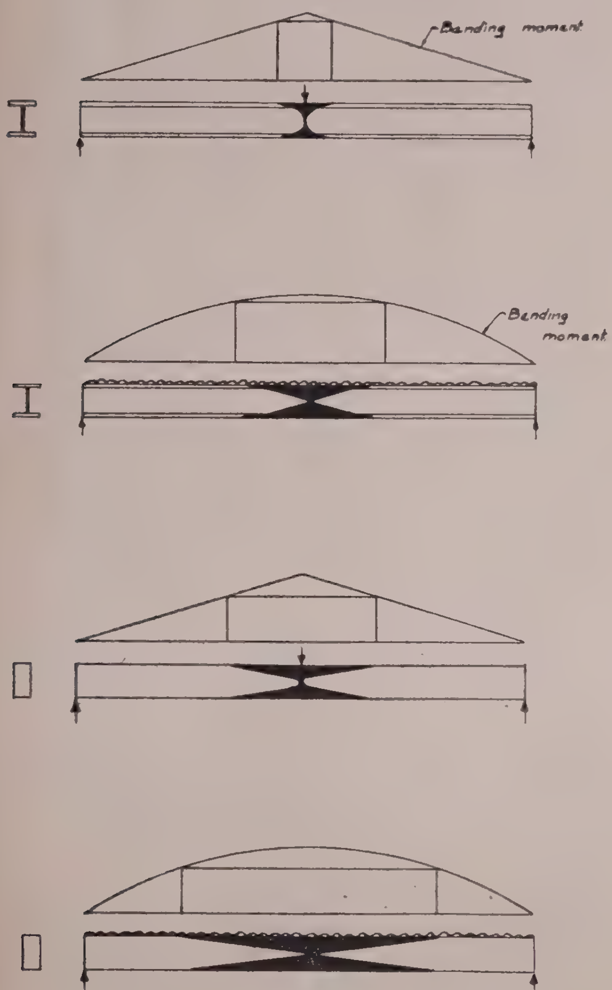

Fig. 1

axis, then the relationship between moment and stress was given by

$$\text{yield stress} = \frac{\text{full plastic moment}}{S}$$

S was also a geometrical property of the section, being in fact the first moment of area about the neutral axis. For rectangular sections $S = 1.5\,Z$, and for joists varied around an average figure of $1.15\,Z$.

" Navier stress " was quite unrelated to the stress developing in the elasto-plastic condition, and its use unnecessarily conflicted with the simple and straightforward principles of the Plastic Theory.

It was interesting to note that M. Lazard's experiments indicated that welded built-up beams behave in conformity with the Plastic Theory. It would be still more interesting to know how riveted built-up girders behave.

Mr. Partridge had noted that, in the proposed improvements to the rules issued by the French Ministry of Reconstruction, it was to be suggested that permissible working stresses be raised by certain percentages to take account of the hidden reserve of strength in mild steel beams. In this country our researches suggested that stress design for rigid frames should be abandoned and that such structures be designed instead to a constant load factor. Mr. Partridge illustrated this point with the following table showing the actual load factors for single span beams with different end fixity conditions, designed to 10 tons/sq. in. at extreme flange fibres :—

	Point Load at centre	Uniformly Distributed Load
Simply-supported ...	1.75	1.75
Fixed one end ...	1.97	2.55
Fixed both ends ...	1.75	2.34

The argument was that if a load factor of 1.75 was good enough for the simply-supported beam then it should be good enough for the others.

If then the same beams were re-designed to a constant load factor of 1.75 the following maximum fibre stresses (in tons/sq. in.) under working load would result :—

	Point Load at centre	Uniformly Distributed Load
Simply-supported ...	10	10
Fixed one end... ...	11.25	14.6
Fixed both ends ...	10	13.35

Although the working stresses varied so much and might seem high in some cases, nevertheless all beams had the same margin of safety against ultimate collapse.

Mr. Partridge concluded by expressing his thanks to M. Lazard for making known the results of his researches and added that he had provided much to think about.

Dr. R. H. Wood (Associate-Member) congratulated the author on his paper and said that one of its striking features was the small difference which drilled holes apparently make to the full plastic moment. He asked if the author would indicate how far one might go in drilling the flanges before this observation must break down, in other words, what was the maximum percentage of flange which could be removed without causing a noticeable difference to the carrying capacity ? Also in the case of punched holes, giving a brittle fracture, was the effect the same in the compression flange as in the tension flange ?

The author had drawn attention to what has become known as the " Stüssi effect," namely the fact that in continuous beams there must be a gradual loss of restraint moment at the supports as the adjacent beams become less stiff until finally the carrying capacity of the beam is little different from that of a simple beam. Without debating whether this effect lay within the limits of practical possibility, Dr. Wood pointed out that the work of the Building Research Station was intimately linked with a more generalised concept of the Stüssi effect, namely, what happens when the frame one is studying is considerably influenced by the presence of walls, floors, encasement and the effects of the soil on which it stands. This had sometimes been referred to as a study of the stiffening effects of the cladding of the building, but it was much better referred to as " composite action " of the frame and the cladding since in many cases (e.g., floors and walls) the " cladding " might be of comparable or sometimes considerably greater

stiffness and in the case of soils it might be considerably less.

Fig. 2 depicted a test of a floor in the new Government Offices, Whitehall Gardens, where the Building Research Station had placed some 160 permanently installed vibrating wire strain gauges for measuring the complete stress history of part of the building during its construction and its normal occupation. In this particular test 12 tons of cast iron weights were used and it was found that the stresses in the beams were (in this case) only about 1/6 of that which the designer would have calculated. Generally speaking, continued Dr. Wood, this meant that there might be very large errors if the frame were studied on its own. The extent of this error depended very considerably on the actual circumstances and no doubt it was an extreme case.

Fig. 2
(*Crown Copyright Reserved*)

The reason was not hard to find, for it was clearly bound up with the actual loads which the frame received, and one of the outstanding problems of elastic or plastic design of multi-storey buildings was to know what loads to apply to the frame. Thus in Fig. 3 was shown the theoretical distribution in the elastic range of vertical intensity of load on the beams supporting a square reinforced concrete slab, when the slab carried uniform live load. The triangular (dotted) distribution was that assumed by the Code of Practice C.P. 114 and it should be realised that this was what would normally be used by the designer and *which he would multiply by a load factor for the purpose of frame design.* This was sufficiently close to the theoretical curve for infinitely stiff beams ($\gamma = \infty$). Here the ratio γ expressed the relative beam to slab stiffness in the form $\dfrac{EI}{DL/_2}$ where

EI = flexural rigidity of beam
D = Flexural rigidity of slab per unit width
L = span.

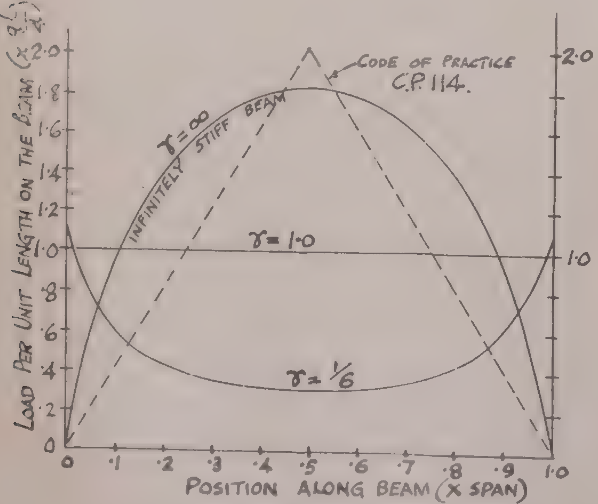

Fig. 3
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Fig. 4
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When the ratio γ became equal to unity there was uniformly distributed load on the beams (approximately) and with beams of smaller stiffness the *load was removed from the centre of the beam* where it counted the most. The effect on the beam bending moments could be quite profound. The problem facing the designer was

Fig. 5
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(a) which of these load distribution "pictures" is present at the working loads.
(b) into what kind of "picture" would this change if plasticity intervened in either the beam of slab.

For example, if the beam were to go plastic first the effect would be to remove still more load from the centre of the span.

Continuing, Dr. Wood said that present indications are that a combination of plastic design for beams with plastic ("fracture line") design of slabs gives good prospects of forecasting the collapse load of the complete system. Thus Figs. 4 and 5 showed the collapse of a square floor with supporting beams and he pointed out that plastic hinges had developed at the centres of each of the four beams with "fracture lines" right across the centre of the slab in both directions. But it was known that the fracture line system for a simply supported slab, with infinitely stiff beams, is a *diagonal* pattern. Extended limit analysis revealed that there is a critical ratio given as follows :—

$$\text{Let } \gamma_P = \frac{\text{(Full plastic moment of beam)}}{\text{(Limiting moment of slab per unit width). } L}$$

$$= \frac{M_B}{M_S.L}$$

When $\dfrac{M_B}{M_S.L} < 1$ then the rectangular mode shown in the photographs applied. (This was deliberately chosen so as to be slightly less than unity).

When $\dfrac{M_B}{M_S.L} > 1$ theory indicated that there should be a sudden switch to a diagonal mode which ignored the beams altogether.

The importance of this was that in the latter case it would be impossible to ascribe a load factor at all to a bare frame design, and in both cases the problem of the "load picture" was overcome by a study of the complete system. This method of design was as yet in its infancy and many more tests were required.

This phenomenon was closely allied to a study of combined collapse modes of portal frames and of the footings. Thus the composite system of frame and footings (as shown by Dr. G. G. Meyerhof, of the Building Research Station) gave rise to collapse modes hitherto unknown in which there were only two, or possibly only one hinge (and in the diagonal mode referred to one might almost say there were no hinges). In this manner it was seen that a study of composite behaviour is analogous to the effects of the adjoining parts of the structure (as noted by Professor Stüssi) but in a more generalised sense. Dr. Wood emphasised that this must not be taken to imply any criticism whatsoever of the "plastic" design of bare frames as such, but rather to point out the difficulty of deciding what were the loads on the frame without a study of the composite behaviour of the structure in the vicinity.

Dr. M. R. Horne said that in discussing the behaviour of rolled steel joists in the plastic range, M. Lazard had quite rightly pointed out that the simple plastic theory can only be regarded as an approximation. It was true and well known that the process of plastic deformation in mild steel is discontinuous. Moreover, the assumption made in the simple theory that almost infinite deformation can take place at constant stress could not be fulfilled in practice. If, however, allowance were made in the theory for strain hardening, it was possible to get extremely close correlation between the results of bending tests on rolled steel joists and tension tests made on specimens taken from the same joists. This had been done at the Engineering Laboratory, Cambridge University, by Dr. J. W. Roderick.

In Fig. 6, Dr. Horne showed the yield stresses obtained in tension tests on material taken longitudinally from

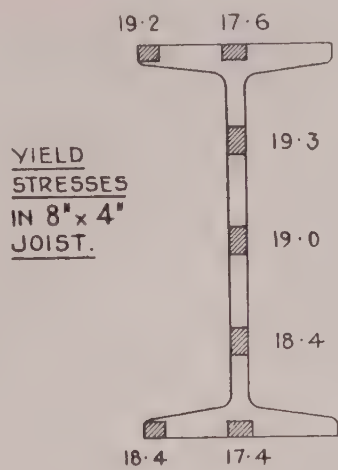


Fig. 6

various places in the cross-section of an 8 in. × 4 in. British Standard R.S.J. He pointed out that the yield stress was almost constant except in the middle of the flanges, where it had a value 7 per cent. lower than in the rest of the cross-section. The shape of the stress-strain curve varied very little over the cross-section and in Fig. 7 he showed the average shape obtained.

Continuing, Dr. Horne said that tests were performed on simply supported joists, both with a single load and

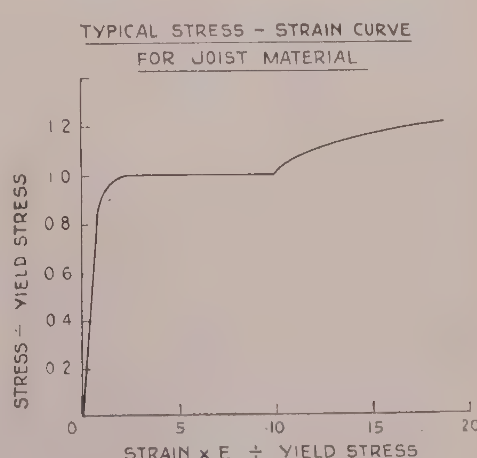


Fig. 7

with two-point loading as shown in Fig. 8. The curves gave a comparison between the experimental deflections and the theoretical deflections obtained from a complete analysis which took strain-hardening into account, from which it would be seen that excellent agreement was obtained. The joists were coated with resin, cracks in which indicated the extent of the plastic zones. Fig. 9 (a) showed the cracks in the web at one stage of the test, the

continuous line being the theoretical boundary of the plastic regions. The illustration showed that agreement was as good as might be expected, taking into account the discontinuous nature of plastic deformation. The conditions at a later stage of the test were shown in Fig. 9 (b).

Dr. Horne said he was interested in M. Lazard's reference to possible modifications in building regulations in view of recent work on the behaviour of steel

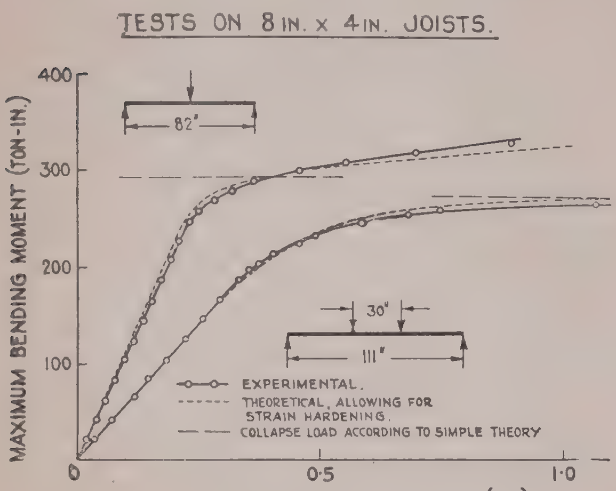


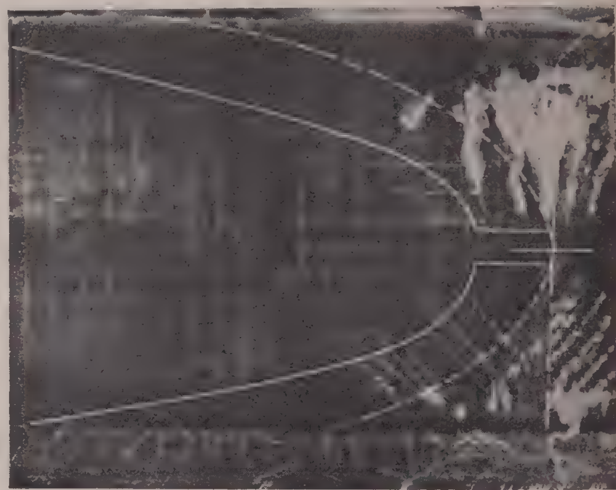
Fig. 8

structures in the plastic range and he agreed with the author's opinion that the question of limiting deflections had to be treated separately from that of limiting strength. He and his colleagues at Cambridge, however, were completely opposed to M. Lazard's suggestion that logical outcome was to make an overall increase in permissible stresses, while retaining existing elastic methods of design. Dr. Horne said he could not speak with any knowledge of French practice, but he was quite sure that to propose increases in working stresses in British practice, after the increases already made in recent years, would be a dangerous step. Existing methods of design, in conjunction with design live loads as at present assumed, were known to give safe structures. We did not know, however, the extent to which we are already really making use of the strength of even statically determinate structures beyond the yield point in supporting occasional overloads in certain cases. To raise working stresses arbitrarily in all structures, including those which are statically determinate, would be a dangerous procedure. Continuing, Dr. Horne said that what we did know was that, if statically determinate structures as at present designed, were safe, then indeterminate structures were unduly conservative. Advantage could only be taken of this fact provided a plastic method were adopted. This was illustrated in Fig. 10, which showed the load-deflection curves for a simply supported beam compared with those for three indeterminate structures. These curves were plotted such that the load to produce first yield was at the same height on the graph. Whatever working stresses were used in an elastic design method, the working loads would be some constant proportion of this yield load, and would therefore be the same in all cases. The real carrying capacities of these structures differed widely, as could be seen from the curves. In order to use this reserve of strength without producing some structures which might be unsafe, design should be based on the collapse loads as given by the upper broken lines. Extensive research had shown that, for a large number of rigid jointed structures, these collapse loads were given with an accuracy quite sufficient for practical purposes by the plastic theory.

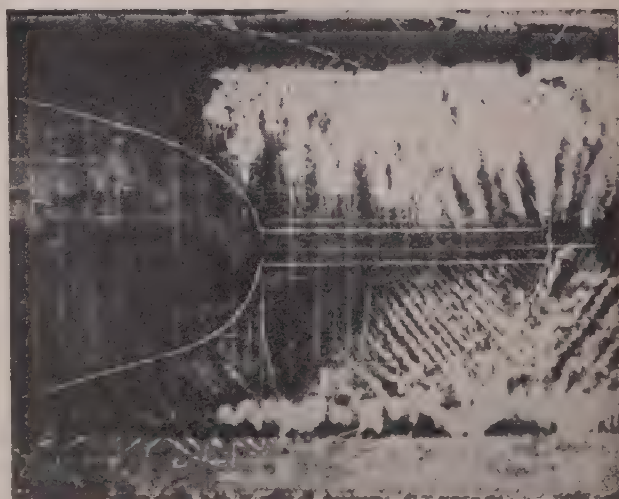
Reply to the Discussion

M. Lazard replies : I must thank first Mr. Terrington for having put my paper in good English.

I agree with him that we must take great care of buckling of flanges and webs and of lateral buckling,



(a)



(b)

Fig. 9

because plastic theoreticians assume generally that buckling is prevented, but do not say how.

I related the Navier stresses of joists with holes both to yield stress and ultimate strength because, as said, it was not possible in my tests to affirm with certitude which was the principal characteristic : I hope that other tests will be done with larger joists to determine this. However, my opinion is that for drilled holes we should probably relate the gross Navier stress to the yield stress and for punched holes we should relate the net Navier stress to the ultimate strength. I should add that the French Contractors' Association do not agree with me on that point, as you can read in the Final Report of the Fourth Congress of the International Association for Bridge and Structural Engineering, page 131, the discussion of my paper by M. A. Dunoyer.

I quite agree with Mr. Terrington that the actual use of a beam will be and must be within the elastic range ;

but that applying a permanent deflection, if small enough, will not reduce the margin of safety ; as shown in Fig. 22, with normal working loads the behaviour will be within the elastic range from a to A. An unexpected very large increase of moving loads (live loads) should happen before reaching plastic yield in B, and even a permanent set in C. What happens now ? It depends on the magnitude of this permanent set. If it is small it might be accepted both by architects and owners. With normal moving loads the beam behaves then from e to E, within a new elastic range : that is, new deflections are proportional to loads and become null with them. If moving loads accidentally exceed their normal values the beam behaves elastically until point C is reached. If moving loads exceed this value—the greatest value ever obtained—a new permanent set will happen. Authorities will be warned and will take due precautions to ensure security, but pending the completion of remedial measures there is no tremendous

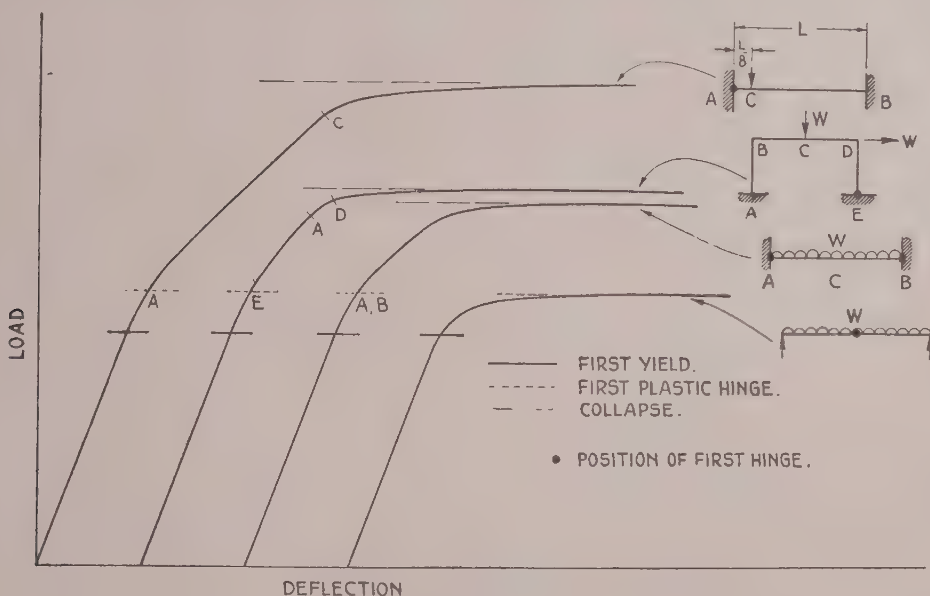


Fig. 10

danger to the structure. This, however, would not be the case if a sudden fracture could occur.

In brief, an accidentally very large excess of moving loads with plain joists will give a warning, and there will be no danger. The margin of safety is great. But I agree with Mr. Terrington's last words, that increases of permissible stresses are not permissible for all types of structures : it depends on the use and on the type of loading.

In reply to Mr. LEWIS, under present economic conditions for French contractors, riveted construction is more general in France. Welding has been and is used, but this use is extending very slowly.

I must point out that welding process needs great care with big structures such as bridges, and we must not forget the Wilson's tests.

Until now, no attempt to use high tensile steel bolts has been made in France to my knowledge. But my office follows their developments in U.S.A. and in U.K.

All the data concerning yield stress, ultimate strength elongation at rupture will be found in my papers ref. 3, 4, 5. I suppose that the rolling process is the same as everywhere. We normally use Thomas steel without any prescription for the chemical composition, except—on account of the Ministry of Public Works Regulations for welded bridges, where we generally use Martin steel with the following restrictions :—

	%
Carbon	$\leq$ 0.20
Silicium	$\leq$ 0.30
Manganese	$\leq$ 1.20
Molybdene	$\leq$ 0.20
Phosphore	$\leq$ 0.04
Soufre	$\leq$ 0.04
Phosphore + soufre ...	$\leq$ 0.07

For general purposes we do not use the Charpy test, nor determine the low temperature notch brittleness, which is of particular use for welding. Our Welding Institute studies this question, and I follow the U.S.A. present tests undertaken under the management of the Welding Research Association in Columbia University.

In French railways we generally avoid building "completely monometallic" welded structures. From place to place, we use riveted joints. Last September I visited, with great interest, the Abington Laboratory, which Mr. Lewis showed us on the screen. I admired this work, especially the many clever processes of erection employed.

In reply to Dr. R. HORNE, I distinguish two parts in his contribution : one dealing with experiments, the other with permissible stresses.

His Fig. 6 gives an example in which the differences between the yield stresses from various places in a joist are not too big. But I am sure that if many other tests are carried out with various joists he will find greater differences. My papers ref. 3 and 5 (page 129) give numerous data with French joists.

Professor Massonnet, of Liege, arrived recently at the same conclusion with Belgian joists (work not yet published).

His Fig. 8 gives a very good agreement between experiment and computation allowing for strain hardening. But I am afraid I am not completely convinced : first, it seems to me, as I said before, difficult to have

generally a good average shape for the stress-strain curve; secondly, as said on page 8 of my paper (II/6/c), to my knowledge the proof has never been given till now of actual strain hardening (I mean a change in yield stress or ultimate strength) in a progressive increase of loading test; nevertheless it could be true.

I follow Dr. Horne when, referring to his Figs. 9a and 9b, he says "that the agreement is as good as might be expected taking into account the discontinuous nature of plastic deformation." My object was only to point out this discontinuous nature "before a whole space is made plastic, so that plastic yield develops later on more equally and isotropically."

On the subject of permissible stresses in statically determinate structures Dr. Horne is opposed to a new increase "after the increases already made in recent years in British practice." I claim only such an increase in French building and bridge regulations where no increase had ever been made. But, contrary to Cambridge views, I think we must go on with the elastic methods of design, knowing on the other hand the true ultimate plastic load, because the behaviour of a structure is in practice elastic and remains elastic even if a small permanent set actually develops (see my reply to Mr. Terrington).

I agree with Dr. Horne that redundant structures are generally "unduly conservative." That is why in French building regulations we introduced the Dutheil's rules to correct the ordinary elastic design method, so that in France it will not be correct in his examples to say that "the working loads would be the same in all cases." On the contrary, it is true with bridges, but till the effect of moving loads on a continuous beam is better known it seems unsafe to try to modify the present regulations.

My object was only to point out that in some cases, that seem to me sufficiently determinate, the plastic theory gives too large an ultimate strength, and that in all cases permanent hinges occur when plastic yield is reached which cannot be accepted for all uses.

I thank Mr. Partridge for his comments, which allow me to give better explanations of some of the points discussed.

In part II, 3, I dealt only with holes in flanges. There have been a few experiments with holes in web: in effect holes seem not to have modified the ultimate load. But I think that if holes have notches, especially when they are located in the flange-to-web junction, we must fear brittle fracture. Such a case happened in a test made by French Railways; it was the starting point of all our investigations in the field of plastic yield in joists and beams.

I think there are no contradictions between paragraphs (f) and (g) of II, 1. In (f) I dealt with failure in a qualitative way, while in (g) I adopted a quantitative one. "Theoretical value" applies to the theoretical full *plastic* moment; that is to Fig. S (see II, 10.a). You will find many references in 3, 4, 5; especially in 3, where you will find some British tests (Hendry's tests). (See also my reply to Mr. Lewis.) I agree with Mr. Partridge that with our ordinary understanding of the Plastic Theory the full plastic moment will only be about 3½ per cent. higher if the yield point for the metal of the web is 15 per cent. higher than that of the flanges; that is why I wrote "In fact the part taken by the web seems to be higher." That is also why, in paragraph (e), I pointed out that the compression stresses acting on the surfaces parallel to the axis may modify the theoretical distribution of the normal stresses. So what can be the true reason? I think that as the yield point varies gradually in a flange from one point to another, and as authors generally give for the yield point only one value that is not the mean value, or perhaps the highest value, the ordinary computation of the theoretical full plastic moment is less than the true one. Are there yet other reasons for difference between actual ultimate bending moment and theoretical ordinary value? That is a question which I cannot answer.

The last words of paragraph (g) attempted to explain differences which have sometimes been found in tests with one point loading (for instance, Roderick's tests).

I think that the plastic failure conditions are not reached when full plasticity has developed at a unique transverse section. However, Fig. 9 (a) and 10 of Horne's paper (Roderick's experiments) show a narrow channel in the elastic range around the neutral axis that seems to be interrupted just at the transverse section under the point of loading. Does this channel remain till the plastic failure?

I do not agree with Mr. Partridge about the utility of the notion of "Navier stress." It enables the designer to compare the bending moments, to know immediately if the yield point is overpassed and by how much, and to use the load factor. For instance, the two tables of his paper may be summarised in the following table giving data of the ultimate Navier Stresses in tons/sq. in.

	Point load at centre	Uniformly distributed load
Simply supported	17.5	17.5
Fixed one end ...	19.7	25.5
Fixed both ends ...	17.5	23.4

You can deduce the maximum fibre stresses when you have chosen the margins of safety (in British practice a safety factor or a load factor) and then be sure that yield point is not exceeded.

In reply to Dr. R. H. Wood, for the first question, further support for what I wrote in II, 5, will be found in my paper under ref. 5 (especially table II), which has just been published. We experimented with beams having nine sections with four drilled holes each and I think this is the maximum percentage of flange which can be drilled in normal practice.

On the subject of punched holes, it is not easy to answer, as we experimented with oscillating moments: the sudden crack started in a hole, which was, at that time, in tension flange; but which had been in preceding cycles, in compression.

The information given by Dr. Wood on what he calls an extension of the Stüssi effect is of the greatest interest. I am glad that his comments go beyond the restricted scope of my lecture, but he will excuse me if I do not answer here. I will only refer to the last words of Dr. Wood, in which he points out the difficulty of deciding, in some cases, what are the loads on the frame without a study of the composite behaviour of the structure in the vicinity.

The valuable works of the Building Research Station, that I visited too quickly last September, are not sufficiently known in France, and I am sure that many French people will be glad to hear Dr. Wood or another of Dr. Thomas's assistants on the subjects of floors, frames and influences of the vicinity: walls, encasements and soils. I should be glad if I could help to arrange a lecture in Paris.

Institution Notices and Proceedings

ORDINARY GENERAL MEETING

An Ordinary General Meeting of the Institution of Structural Engineers was held at 11, Upper Belgrave Street, London, S.W.1, on Thursday, December 17th, 1953, at 5.55 p.m. ; Lt.-Colonel R. F. Galbraith, M.C., B.Sc., M.I.Struct.E., A.M.I.C.E. (President), in the Chair.

The following members were elected in accordance with the Bye-Laws. Will members kindly note that the elections, as tabulated below, should be referred to when consulting the Year Book for evidence of membership.

Students

Ang Kin Lian, of Singapore.
Atkinson, Angela Wendy (Miss), of Harrogate, Yorks.
Bradbury, Barry James, of Prestwich, Nr. Manchester.
Burnell, Ian Derek, of Loughton, Essex.
Lawton, Keith, of Timperley, Cheshire.
Nadarasa, Mailvaganam, of London.
Parr, John, of Prescot, Lancs.
Sehmi, Baldip Singh, of London.
Traverse, John Derek, of St. Helens, Lancs.
Yardley, Laurie, of London.

Graduates

Backhouse, Samuel Roy, A.M.I.C.E., of Millerston, Glasgow.
Bickerton, David James, B.Sc.(Civil) Bristol, of Bristol.
Bradstock, John Walter, B.E.(Civil) New Zealand, of London.
Bridge, Stuart Berry, of Glossop, Derbyshire.
Bruce, John Nice, of Biggin Hill, Kent.
Case, Lionel Francis, of London.
Cheng Hon Kwan, of Hong Kong.
Clements, Arthur Charles, B.Sc.(Eng.) London, of London.
Davies, John Duncan, B.Sc.(Eng.) London, of Swansea.
Deshpande, Vasant Balkrishna, of Poona, India.
Dilks, John Roxey, B.Sc.(Civil) Durham, of Blyth, Northumberland.
Ding Gar Day, B.Arch.N.Z., of Dunedin, New Zealand.
Edgeley, Peter, of Sheffield, Yorks.
Firth, John Albert, of Brighouse, Yorks.
Goleniewski, Stanislaw, of London.
Harrison, Henry Peter, A.R.I.B.A., of London.
Henderson, Leslie Nathan, of Stockport, Cheshire.
Hughes, John George, of Rugeley, Staffs.
Hussain, Munawar, of Lahore, Pakistan.
Jones, Kenneth Stuart Leslie, of Stafford.
Keller, George Charles Chapman, of Cape Town, South Africa.
Kwaku, Samuel Francis, B.Sc.(Civil) Bristol, of London.
Lambert, John Denis. B.Sc.(Eng.) London, of Birmingham.
Lowson, William Wallace, B.Sc.(Eng.) Glasgow, of Glasgow.
Maisel, Bruce Isaac, B.Sc.(Civil) Rand, of Johannesburg, South Africa.
Marsh, Kenneth Ormrod, of Redcar, Yorks.
Mills, Gordon Ernest, of Paeroa, New Zealand.
Norman, Alan Geoffrey, B.Sc.(Eng.) Nottingham, of Beeston, Notts.
Parsons, Geoffrey Frank, of London.
Pennington, Derek Alan, of Salford, Lancs.
Pontin, Ronald Frederick, of London.
Rowcroft, Alan Roy, of Melksham, Wilts.
Richardson, Paul Herbert, of Jamaica, B.W.I.
Sharples, Kenneth, of Bolton, Lancs.
Singh, S. Balbir, of Hirapur, Shanbad, India.
Srikantia, Santhebachalli, B.Eng. Mysore, of Bombay, India.
Venkatarama Reddy, Dronnadulla, B.E.(Civil) Madras, of London.
Weddell, Thomas William, B.Sc.(Civil) Durham, A.M.I.Mun.E., of Sunderland, Co. Durham.
Wood, John Frederick Dilworth, of Liverpool.

Associate-Members

Balachandran, Nelliet Koroth, B.E.(Civil) Madras, of Madras, India.
Chandramouli, Tunuguntla, of Madras, India.
Chokhavatia, Professor Ratilal Harilal, B.Sc.(Civil) Bombay, of Gujarat, India.
Clugston, Douglas Walter, B.Sc.(Civil) Glasgow, of Glasgow.
Fairman, Charles Edward John, of Salisbury, Southern Rhodesia.
Karanjia, Minocher Rustomjee, B.E.(Civil) Bombay, of Bombay, India.
Yue Shun-Kwong, of Hong Kong.

Members

Cooke, Louis, of Huddersfield, Yorks.
Whalley, Alfred Whitaker, of Bowdon, Cheshire.

Transfers

Students to Graduates

Baker, Derek William, of London.
Booth, William Harold, of Calgary, Alberta, Canada.
Case, John Kenneth, of London.
Chapman, William Edwin, A.M.I.C.E., of Hope, Nr. Sheffield, Yorks.
Dampier, Bernard William, of Manchester.
Harris, Colin Frederick, of Kampala, Uganda, East Africa.
Jones, Henry James King, of Newbury, Berks.
Little, Derek Sidney, of Romford, Essex.
Pugh, Trevor George, of Auckland, New Zealand.
Rutherford, Michael William, of Pretoria, South Africa.
Sowerby, Paul Leon, of Manchester.
Strachan, Alan, of Altrincham, Cheshire.
Thistle, Sydney Alan, of Johannesburg, South Africa.
Yeo Lim Hui, of Singapore.

Graduates to Associate-Members

Bennington, Kenward, B.Sc.(Civil) Cape Town, of Cape Town, South Africa.
Bolingbroke, Kenneth, of Brighton, Sussex.
Cragg, Wilfred Shipham, of Pinelands Cape, South Africa.
Evans, Peter Robert, of Porthcawl. Glam.
Halford, Frederick Samuel, M.C., of Salisbury, Southern Rhodesia.
Krarup, Anthony Charles, B.Sc.(Eng.), London, of Chilwell, Notts.
Lewis, Lawrence Henry, of Ewell, Surrey.
Louw, Timotheus, B.Sc.(Civil) Rand, of Pretoria, South Africa.
Mahfouz, Georges Elias, B.Sc., of Cairo.
Morley, Kenneth Outram, B.Sc.(Civil) Cape Town, of Port Elizabeth, South Africa.
Nichols, John Russell, B.Sc.(Eng.) Cape Town, of Germiston, Transvaal, South Africa.

PINTO, Vitorino Antonio C. L., B.E.(Civil) Bombay, of Goa, Portuguese India.
RATHOD, Mahindra Valjibhai, B.E.(Civil) Bombay, of Bombay, India.
ROBINSON, William Crathern, of London.
WALKER, Eric Stuart, B.Sc.(Eng.) Glasgow, of Bulawayo, Southern Rhodesia.

Associate-Members to Members

GORDON, Leslie, B.Sc. Edinburgh, M.I.C.E., of Edinburgh.

Members to Retired Members

ADDIE, James, of Bexhill-on-Sea, Sussex.
BOUCHER, Cyril Lett, M.I.C.E., A.M.I.Mech.E., of Birmingham.
BRODRICK, Henry, of Ewell, Surrey.
FITT, William James, O.B.E., F.R.I.B.A., of Bournemouth.
GRANTHAM, George James, L.R.I.B.A., of Birmingham.
GREEN, Douglas Harold, O.B.E., M.C., B.Sc.(Eng.), A.C.G.I., M.I.C.E., of Bournemouth.
JAMES, John Thomas, of Lincoln.
MAWSON, Edward Prentice, F.R.I.B.A., of Lancaster.
MAYHEW, Alfred Ernest, A.R.I.B.A., of Wembley, Middlesex.
NOWLAN, Horace John, I.S.O., M.I.C.E. of Worthing, Sussex.
PEACOCK, John, of Barnton, Midlothian.
REYNOLDS, Frank Stephen, of Lostwithiel, Cornwall.
SIMPSON, Alexander McKay, of Kenley, Surrey.

RE-ADMISSIONS

Associate-Membership

CARTWRIGHT, Herbert Edward, of Llanddulas, North Wales.
SOHAL, Sardar Kartar Singh, of Jamananagar, Jagadhari, India.
WILLIAMS, John Thomas, of London.

OBITUARY

The Council regret to announce the deaths of K. W. BRANCZIK, Stelio MACERATA, William MUIRHEAD,* Robert USHERWOOD (Members); George Reginald DAVIS, Captain Kenneth Noss ARNOLD (Retired Members); Thomas CAMPBELL-GRAY (Associate Member).

*Founder-Member.

RESIGNATIONS

Notification was given that the Council had accepted with regret the resignations of Horace Raymond Chanter, James Bushell Hutchins, George Orchard (Members); Charles Frederick Barker, George Tolley (Retired Members); Yeo Soy Bah (Associate); Benjamin William Bays, Charles Hubert Biddulph, Leonard Birch, Eric Colin Hall, Francis Lynn Wake (Associate-Members); Leonard Stanley Davey, Thomas Louis Farnes, James William Hoseason (Graduates); David John Balaam, Robert Slater Bowen, John Michael Curran, Robert Norman Dickinson, Donald Franklin Dyson, Kenneth Alfred Parrish, Norman Walter Shiner, Onkar Dayal Shrivastava, John McAinsh Thomson (Students).

EXAMINATIONS, JULY, 1954

The Examinations of the Institution will next be held at Centres in the United Kingdom and Overseas on July 13th and 14th, 1954 (Graduateship), and 15th and 16th (Associate-Membership).

OVERSEAS REPRESENTATIVES

The Council have appointed Dr. K. Billig, M.I.C.E. (Member), to be the Institution's Representative in Roorkee, India, and Mr. C. Dupenois, B.Sc. (Associate-Member), to be the Institution's Representative in the British West Indies.

DRURY MEDAL AWARD, 1953

The Council have awarded the Drury Medal for 1953 to Mr. G. W. E. FEAKES,(Student), of Reading.

The Competition is held biennially and the subject set for this, the fourth Competiton, was the design of the structure of a new factory building.

Letters of Commendation have been awarded to Mr. T. K. HOUGHTON (Graduate), of Stockport, Cheshire, and Mr. C. L. H. WEAVER (Graduate), of London.

The work of Mr. T. A. Clark (Graduate), of Chigwell, Essex, and Mr. S. W. Knott (Graduate), of Manchester, has also been commended.

MACLACHLAN LECTURE COMPETITION

The closing date for the receipt of entries for the MacLachlan Lecture Competition is Wednesday, March 31st, 1954. The General conditions of the competition are as follows:

1. The Institution of Structural Engineers shall institute a written lecture to be known as the MacLachlan Lecture and to be held annually.

2. The subject of the Lecture may be on any aspect of Structural Engineering as long as in every second year the subject shall be confined to steel structures.

3. Entrance into the competition for the Lecture shall be confined to Associate Members of the Institution, who are under the age of 32 years.

4. All papers entered for the competition shall be submitted to assessors to be appointed by the Council of the Institution, and all such papers (including the prize-winning Lecture) shall be available for publication in the Journal of the Institution at the discretion of the Council.

5. No paper submitted shall have been published or read elsewhere.

6. The winner of the competition shall be required to present the Lecture to a meeting of the Institution at which he will be presented with the sum of £17 10s.

7. Should a competitor's paper be considered worthy of ranking second in merit he will receive a consolation award of £5.

8. In the event of there being no winner of the competition in any one or more years, whether because no lecture submitted is considered to be of sufficient merit to warrant award, or for any other reason, the Institution shall transfer the above sums to the Research Fund of the Institution.

PARTICULARS OF THE COMPETITION FOR 1954

1. The MacLachlan Lecture will be given at a meeting of the Institution to be arranged towards the end of 1954.

2. The subject of the Lecture shall be on any aspect of structural engineering.

3. The work should be submitted as the script of a lecture which the author, if successful in the competition, will deliver before an audience in the course of about one hour. The development of mathematical formulæ and detailed calculations should be avoided as far as possible in the text; if they are essential they should be embodied in appendices. Photographs, drawings, graphs, etc., which would appear as illustrations to the lecture in published form, should accompany the script. If additional illustrations would be shown as slides, a list of these should be included.

4. Six copies of each Lecture should be submitted and should be addressed to the Secretary of the Institution.

5. The closing date for the receipt of entries by the Institution is Wednesday, March 31st, 1954.

FORTHCOMING MEETINGS

The following meetings will be held at 11, Upper Belgrave Street, London, S.W.1.

Thursday, February 25th, 1954

Ordinary General Meeting, for the election of members, 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when Mr. R. A. Sefton Jenkins, B.Sc., A.M.I.C.E., A.M.I.Struct.E., A.C.G.I., will give a paper on "Prestressed Steel Lattice Girders."

Thursday, March 11th, 1954

Ordinary Meeting, 6 p.m., when Mr. S. J. Crispin, M.I.Struct.E., L.R.I.B.A., will give a paper on "Soil Stabilisation in Fine Materials."

Thursday, March 25th, 1954

Ordinary General Meeting, 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when Dr. P. B. Morice, B.Sc., and Mr. G. Little, M.Sc., will give a paper on "Load Distribution in Prestressed Concrete Bridge Systems."

Thursday, April 22nd, 1954

Ordinary General Meeting, 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when Dr. R. Weck will give a paper on "Fatigue of Welded Structures."

Members wishing to bring guests to the Ordinary Meetings announced above are requested to apply to the Secretary for tickets of admission.

SUMMER MEETING

The Summer Meeting of the Institution will be held in Birmingham from May 18th to May 21st, 1954. A Civic Reception will be given by the Lord Mayor of Birmingham on the evening of Tuesday, May 18th. On Wednesday morning, May 19th, a technical paper will be given, and on Thursday evening, May 20th, a Banquet will be held at the Grand Hotel.

Details of the programme are being arranged and will be announced in future issues of the Journal.

EXAMINATIONS

Preparation for the Examinations of the Institution by Attendance at Technical Colleges

A candidate for Graduateship or Associate-Membership may be able to attend a technical college; these notes are intended to guide him in choosing the most suitable instruction.

Preparation for the Graduateship Examination

Technical Colleges offer:

(a) Full-time courses for degrees of Higher National Diplomas in Building or Engineering.

(b) Part-time day or evening courses for Higher National Certificates in Building or Engineering.

If he obtains a Higher National Certificate or Diploma complying with Appendix II, Section V, of the Regulations Governing Admission to Membership, the candidate will be exempted from the Graduateship Examination.

Alternatively, he may study subjects selected from the available courses and sit the Graduateship Examination. At technical colleges courses are usually available in Building Science or Engineering Science, Strength of Materials, Theory of Structures and Surveying, but students are not normally allowed to select subjects from National Diploma or Certificate courses unless they can show evidence of sound training in more elementary studies. The advice of the College Authorities should be followed.

Preparation for the Associate-Membership Examination

At some technical colleges there are part-time courses in Structural Engineering which cover the syllabus of the Associate-Membership Examination. At other colleges the candidate must rely on Higher National Certificate courses or on advanced courses in Building, Civil Engineering or Municipal Engineering; these cover only part of the requirements for the Associate-Membership Examination.

Colleges in List "A" provide at least two years of instruction in Theory of Structures and in Structural Engineering Design and Drawing up to Associate-Membership standard. They also give instruction in Structural Specifications, Quantities and Estimates.

List "A"

- Belfast College of Technology.
- Birmingham College of Technology.
- Bolton Municipal Technical College.
- Bradford Technical College.
- Bridgend Technical College.
- Chesterfield College of Technology.
- Derby Technical College.
- Dudley and Staffordshire Technical College.
- Glasgow Royal Technical College.
- City of Liverpool College of Building.
- L.C.C. Brixton School of Building, S.W.4.
- L.C.C. Hammersmith School of Building and Arts and Crafts, W.12.
- Manchester College of Technology.
- Middlesbrough, Constantine Technical College.
- Nottingham and District Technical College.
- Salford, Royal Technical College.
- South-East London Technical College, Lewisham Way, S.E.4.
- South-West Essex Technical College, Walthamstow, E.17.
- Stockport College for Further Education.
- Twickenham Technical College.
- Willesden Technical College, N.W.10.

Colleges in List "B" provide instruction in Theory of Structures from which the student may reach Associate-Membership standard, but instruction in Structural Engineering Design and Drawing and in Structural Specifications, Quantities and Estimates is not usually so complete.

List "B"

- Brighton Technical College.
- Cardiff Technical College.
- Edinburgh, Heriot-Watt College.
- Huddersfield Technical College.
- Leeds College of Technology.
- London, Battersea Polytechnic, S.W.11.
- London, Northampton Polytechnic, E.C.1.
- L.C.C. Westminster Technical College, S.W.1.
- Plymouth and Devonport Technical College.
- Preston, Harris Institute.
- Wigan Mining and Technical College.
- Woolwich Polytechnic, S.E.18.

Students are advised to take the organised courses in Structural Engineering where these are available.

LONDON GRADUATES' AND STUDENTS' SECTION

The following meetings will be held at 11, Upper Belgrave Street, London, S.W.1.

Tuesday, February 16th, 1954

Address by the President of the Institution, 6 p.m.

Tuesday, March 9th, 1954

Annual General Meeting. Followed by Film, "The Failure of the Tacoma Narrows Suspension Bridge."

Tuesday, April 13th, 1954

Mr. E. M. Lewis, on "Welding Construction."

Hon. Secretary : J. F. S. Pryke, B.A.(Hons.), Bushcroft, Slipe Lane, Wormley, Herts.

BRANCH NOTICES

LANCASHIRE AND CHESHIRE BRANCH

The following meetings have been arranged :—

Thursday, February 11th, 1954

Dr. F. G. Thomas, B.Sc., M.I.C.E., M.I.Struct.E., on "Composite Action in Structures."

Thursday, February 18th, 1954

Annual Dinner and Dance at the Grand Hotel, Manchester. The President and the Secretary will attend.

Wednesday, March 10th, 1954

Joint Meeting with the Liverpool Engineering Society. "The Uses of Aluminium for Structural Purposes," by a member of the staff of the Aluminium Development Association. At the Temple, 24, Dale Street, Liverpool, 6 p.m.

Thursday, March 25th, 1954

Joint Meeting with the Reinforced Concrete Association, North-Western Branch. Mr. G. P. Bridges, M.I.Struct.E., A.M.I.C.E., L.R.I.B.A., on "The Design and Construction of Reinforced Concrete Silos and Bunkers."

Wednesday, April 28th, 1954

Annual Business Meeting, followed by a film show.

All meetings, unless otherwise stated, will be held in the Reynolds Hall, College of Technology, Manchester, at 6.30 p.m., preceded by tea at 5.45 p.m.

Joint Hon. Secretaries : A. S. Sinclair, A.M.I.Struct.E., 17, The Circuit, Cheadle Hulme, Cheshire ; M. D. Woods, A.M.I.Struct.E., 58, Spring Gardens, Salford, 6.

MIDLAND COUNTIES BRANCH

The following meetings have been arranged :—

Tuesday, February 9th, 1954

Mr. E. Shepley, B.Sc., A.M.I.C.E., on "Prestressed Concrete Framework for Liverpool University Medical School." At the Supper Room, The King's Hall (Queen Street Baths), Queen Street, Derby, 7 p.m.

Friday, February 26th, 1954

Mr. C. B. Brewington, B.Sc., A.M.I.C.E. (Graduate), and Mr. J. W. Fortey, A.M.I.Struct.E., A.C.T. (Birmingham), on "A Method of Structural Analysis by Large-Scale Models."

Friday, March 26th, 1954

Mr. H. V. Hill, M.Sc., A.M.I.C.E., A.M.I.Struct.E., on "The Load-Bearing Capacity of Metal Structures."

Friday, April 30th, 1954

Annual General Meeting. Followed by Paper :— "Some Factory Building Maintenance Problems," by Mr. W. T. Dudley, A.M.I.Struct.E.

All meetings, except where otherwise stated, will be held in the James Watt Memorial Institute, Great Charles Street, Birmingham, at 6 p.m.

Hon. Secretary : L. A. Firminger, A.M.I.Struct.E., 656, Chester Road, Erdington, Birmingham, 23.

GRADUATES' AND STUDENTS' SECTION

Wednesday, March 31st, 1954

Address by the Chairman of the Midland Counties Branch, followed by the Annual General Meeting, at the James Watt Memorial Institute, Great Charles Street, Birmingham, 7 p.m.

Hon. Secretary : H. L. Bramwell (Graduate), 139, Wood Lane, Handsworth Wood, Birmingham, 20.

NORTHERN COUNTIES BRANCH

The following meetings have been arranged :—

Tuesday, February 2nd, 1954

Mr. A. J. Harris, B.Sc.(Eng.), A.M.I.C.E., on "Prestressed Concrete in Civil Engineering Works." At Middlesbrough.

Wednesday, February 3rd, 1954

The above meeting will be repeated at Newcastle.

Tuesday, March 2nd, 1954

Professor W. Fisher Cassie, M.Sc., Ph.D., F.R.S.E., M.I.C.E., M.I.Struct.E., on "Pavement Structures," at Middlesbrough.

Wednesday, March 3rd, 1954

The above meeting will be repeated at Newcastle.

Thursday, March 18th, 1954

Ladies' Guest Night, at Middlesbrough.

Friday, March 19th, 1954

Ladies' Guest Night, at Newcastle.

Wednesday, April 7th, 1954

Annual General Meeting, at Newcastle.

All meetings will commence at 6.30 p.m., the Middlesbrough meetings being held at the Cleveland Scientific and Technical Institution, Corporation Road, and the Newcastle Meetings in the Neville Hall, near the Central Station.

Hon. Secretary : O. Lithgow, A.M.I.Struct.E., 4, Stoneleigh Avenue, Acklam, Middlesbrough.

NORTHERN IRELAND BRANCH

The following meetings have been arranged :—

Tuesday, February 9th, 1954

Annual Dinner and Social Function, at the Grand Central Hotel, Belfast, 6 p.m. Visit of the President and the Secretary of the Institution.

Tuesday, March 2nd, 1954

Details to be announced.

Tuesday, April 6th, 1954

Annual General Meeting.

Except where otherwise stated, meetings will be held at the College of Technology, Belfast, at 6.45 p.m., preceded by tea at the Overseas League premises, Wellington Place, Belfast, at 6 p.m.

Hon. Secretary : A. H. K. Roberts, B.A., B.A.I., M.I.C.E.I., M.I.Struct.E., "Barbizon," 26, Dunlambert Park, Belfast.

SCOTTISH BRANCH

The following meetings have been arranged :—

Tuesday, February 9th, 1954

Mr. C. M. Wilson, A.M.I.C.E., A.M.I.Struct.E., on "The Reconstruction of Portobello Power Station." at the Ca'doro Restaurant, Glasgow, 6 p.m.

Thursday, February 25th, 1954

At the Elite Hotel, Edinburgh, 6 p.m., Mr. Hugh B. Sutherland, S.M.(Harvard), A.M.I.C.E., F.G.S.,

A.M.I.Struct.E., on "Some Problems in Foundation Engineering."

Tuesday, March 16th, 1954

At the Ca'doro Restaurant, Glasgow, 6 p.m., Mr. F. R. Bullen, B.Sc., M.I.C.E. (Hon. Curator), M.I.Struct.E., on "Unusual Design for a Large Constructional Shop."

Tuesday, April 13th, 1954

Annual General Meeting. At Ca'doro Restaurant, Glasgow, 6 p.m.

Hon. Secretary : G. Drysdale, A.M.I.Struct.E., "Niaroo," 33, Union Street, Motherwell, Lanarkshire.

SOUTH-WESTERN COUNTIES BRANCH

The following meetings have been arranged :—

Friday, February 19th, 1954

At the Duke of Cornwall Hotel, Plymouth, 7 p.m., Mr. A. V. R. Hooker, M.I.Struct.E., A.M.I.C.E., on "Structural Engineering at Abbey Works, Margam."

Friday, March 19th, 1954

Film on "Bridging," at the Demonstration Theatre of the South-Western Gas Board, Union Street, Torquay, 7 p.m.

Joint Hon. Secretaries : E. W. Howells, M.I.Struct.E., 10-12, Market Street, Torquay, Devon ; and C. J. Woodrow, "Elstow," Hartley Park Villas, Tavistock Road, Plymouth, Devon.

WALES AND MONMOUTHSHIRE BRANCH

The following meetings have been arranged :—

Wednesday, February 17th, 1954

At Swansea. Junior Members' Evening.

Tuesday, March 9th, 1954

At Cardiff. Joint Meeting with the Institution of Civil Engineers. "Barry Dry-Dock Reconstruction."

Friday, March 26th, 1954

At Swansea. Annual Dinner. The President and the Secretary of the Institution will be present.

Wednesday, March 31st, 1954

At Swansea. Mr. S. Woolf, on "Recent Developments in Timber Structures."

Meetings at Cardiff will be held at the South Wales Institute of Engineers, Park Place, at 6.30 p.m.

Meetings at Swansea will be held at the Mackworth Hotel, at 6.30 p.m.

Hon. Secretary : G. R. Brueton, A.M.I.C.E., A.M.I.Struct.E., 86, The Exchange, Mount Stuart Square, Cardiff.

WESTERN COUNTIES BRANCH

Friday, February 5th, 1954

Mr. J. E. Collins, A.R.I.B.A., A.M.I.Struct.E., on "The Industrial Architect and Engineer."

Wednesday, February 17th, 1954

Annual Dinner at the Royal Hotel, Bristol.

Thursday, March 4th, 1954

Combined Meeting with the Institution of Civil Engineers. Details to be announced.

Friday, April 2nd, 1954

Annual General Meeting, followed by a Film Show.

Unless otherwise stated, all meetings will be held in the University of Bristol Geology Lecture Theatre, at 6 p.m., preceded by tea at 5.30 p.m.

Hon. Secretary : E. Hughes, A.M.I.Struct.E., 23, Southdown Road, Westbury-on-Trym, Bristol, 9.

YORKSHIRE BRANCH

The First Ordinary Meeting of the Yorkshire Branch was held on Wednesday, October 14th, 1953, in the Great Northern Hotel, Leeds, at 6.30 p.m., and was well attended. Mr. J. Guthrie Brown, Vice-President, and Major R. F. Maitland, Secretary of the Institution, were present. The President, Lt.-Colonel R. F. Galbraith, was unable to attend owing to indisposition, and Mr. Guthrie Brown tendered his apologies.

Mr. Guthrie Brown then presented a Diploma to Dr. S. Mackey for his paper entitled "An Investigation of the Behaviour of a Riveted Plate Girder under Load," which he had given in collaboration with Dr. D. M. Brotton at a meeting in London. The Vice-President then expressed the appreciation of the Branch to the retiring Chairman, Mr. D. R. S. Wilson, for his services during the past Session, and installed the new Chairman, Mr. John Dossor. Mr. Dossor expressed his thanks to the members for electing him to the Chair, and also thanked Mr. Wilson for his help during the past Session. He then gave his Address.

A vote of thanks was accorded the Chairman, and Major R. F. Maitland then spoke of the work done by the Institution, and of the increasing membership.

Wednesday, February 17th, 1954

Mr. H. D. Morgan, M.Sc., M.I.C.E., on "Driving and Testing of Piles."

Friday, March 12th, 1954

Annual Dinner and Dance, at Parkway Hotel, Bramhope, Leeds, 7 p.m.

Wednesday, March 17th, 1954

Mr. E. Lightfoot, M.Sc., B.Sc., A.M.I.C.E., A.M.I. Struct.E., on "Dynamic Stresses in Structures."

Friday, March 26th, 1954

Joint Meeting with the Yorkshire Association of the Institution of Civil Engineers, in Hull. Professor A. L. L. Baker, B.Sc., M.I.C.E., M.I.Struct.E. (Vice-President), on "Jetties and Fenders."

Wednesday, April 28th, 1954

Annual General Meeting, followed by "The Quality Control of Concrete," by Dr. J. L. Murdock, M.Sc.

All meetings will be held at the Great Northern Hotel, Leeds, at 6.30 p.m., except where otherwise stated.

Hon. Secretary : E. Wrigley, A.M.I.Struct.E., 17, The Drive, Alwoodley, Leeds.

UNION OF SOUTH AFRICA BRANCH

Branch Hon. Secretary : A. E. Tait, B.Sc., A.M.I.C.E., A.M.I.Struct.E., P.O. Box 3306, Johannesburg. During week-days Mr. Tait can be contacted in the City Engineer's Department, Town Hall, Johannesburg. 'Phone: 34-1111, Ext. 257.

Natal Section Hon. Secretary : E. G. Bennett, A.M.I.Struct.E., c/o The Reinforcing Steel Co., Ltd., P.O. Box 49, Merebank, Durban.

Cape Section, Hon. Secretary : R. Stubbs, M.I.Struct.E., The Reinforcing Steel Co., Ltd., P.O. Box 2692, Cape Town.

Book Reviews

Carpentry for the Building Trades, 2nd Edition, by E. A. Lair. (New York and London : McGraw-Hill, 1953.), pp. 310 plus viii, 9 in. × 6 in., 38s.

A new well-revised edition of this practical book has been published, in which there has been some re-arrangement of material to facilitate use. Clear working instructions are given for all carpentry work on small and medium-sized framed houses, particular emphasis being on modern construction. New methods of ventilation and insulation are included and details are given of the latest building materials and of modern methods of estimating.

The book, which is well illustrated and contains useful tables and charts, provides a course for classes in carpentry.

An Engineer's Approach to Corrosion, by C. F. Trigg, M.Sc., A.M.I.C.E., A.M.I.Mech.E. (London : Sir Isaac Pitman & Sons, 1952.), 111 pp. bibliography and Index. 15s.

This work is divided into eight chapters which deal thoroughly with three aspects of corrosion.

Three chapters are devoted to the mechanism of corrosion in iron, steel, concrete and many other materials. Of particular interest to structural engineers are the references to soil corrosion and to corrosion of steel in concrete.

Chapter IV deals with the effect of corrosion on the life of rolled steel sections and other structural members. A method of calculation of the reduced Load Factor due to corrosion is proposed.

The remaining chapters describe methods of corrosion prevention, preparation of materials for corrosion proofing and protective treatments.

The book is illustrated by seven plates and numerous diagrams, graphs and tables, laid out in a clear and concise manner.

It is strongly recommended to all practising engineers and to those studying for a degree of National Certificate in engineering.

D. W. C.

Notch Bar Testing and its Relation to Welded Construction. (London : Institute of Welding, 1953.) v + 79 pp. ; 11 in. × 8½ in. 25s.

The book under review records five papers presented at a Symposium organised by the Institute of Welding in December, 1951, under the Chairmanship of Professor E. C. Rollason. Two of the papers were presented by steelmakers and three by research organisations, the latter setting out the viewpoint of the user. A record of the discussions on the papers is included in the volume under review.

Messrs. W. Barr and I. M. Mackenzie describe notch bar testing and the selection of steel for welded construction, and include in their practical consideration a word of caution that the steel-maker knows better than the user what steel is good for him.

Mr. G. M. Boyd, of Lloyds Register of Shipping, gives an assessment of notch ductility by a variety of notch tests, and provides a precis of the limits of accuracy of the Izod, Charpy, Mesnager and Schnadt as impact tests, and of the Tipper test and the " Navy Tear Test." It is pointed out that discrimination between steels with regard to toughness in service can, in the present state of knowledge, only be made by exercising judgement after taking many circumstances into consideration in the light of experience.

Professor Ing. W. Soete provides a paper on the state of stress and brittle fracture. Dr. Constance Tipper gives a paper on notch bar tests in relation to service performance and Dr. J. H. Van der Veen deals with the development of the testing method of brittle fracture of mild steel plates. Dr. N. P. Allen, who acted as Rapporteur, provided a masterly summary of the Symposium after a number of experts had contributed to the discussion.

A. C. V.

Prestressed Concrete, by Y. Guyon. English translation edited by W. M. Johns. (London : Contractors' Record and Municipal Engineering, 1953.) xv plus 543 pp. 10 in. × 6¼ in. 70s.

This important work was first published in French in 1951 by Ed. Eyrolles and a comprehensive review appeared in THE STRUCTURAL ENGINEER, Vol. XXX, No. 9, September, 1952, pp. 223-4. As a result of the considerable English-speaking interest, an English edition has now been published, which, it is stated, is a free translation containing minor changes and additions to broaden the appeal.

The book, which is divided into three parts, is restricted to statically determinate straight beams, the aim being to bring out the general principles involved in prestressing by a fairly detailed study of some particular applications, and to describe some methods which can be suitably modified for other applications.

The first part deals with prestressing methods and plant, materials used, friction between cable and duct, fire resistance, anchorage zone stresses in post-tensioned beams, anchorage by bond and anchorage zone stresses in pre-tensioned beams. The elastic design of statically determinate straight beams is the subject of the second part and in the third part, the author described three large-scale tests on post-tensioned beams and a number of tests on smaller pre-tensioned beams with bonded wires, and cracking tests on rectangular beams. This section ends with a summary of results and a discussion of factors of safety and elasto-plastic design.

Three Appendixes on anchorage zone stresses in rectangular beams, on direction of shear cracks in prestressed beams and on distribution of shear stresses in beams of variable depth complete the volume.

How to Draw Perspectives to Scale, by W. H. Fuller. (Studio Publications, 1952.) 6¾ in. × 5½ in. 64 pp. 3s. 6d.

In this small volume the author sets out to demonstrate methods of producing scaled perspective drawings, and he concentrates on " how " rather than " why." The diagrams are well drawn and the reader who is possessed of the necessary draughtsman's skill will find the simple step-by-step pictorial instructions very useful in the determination of the essential lines and points. He should then have no difficulty in producing satisfactory perspective drawings.

There are some minor defects in presentation, for example the " measuring line " method is illustrated in Figs. 9 and 14, but its use is not explained until Fig. 16, which occurs several pages further on. However, the whole book gives a very condensed account of practical methods and should be useful to anyone requiring to produce perspective illustrations for any purpose.

P. E. S.

Soil Stabilisation in Fine Materials*

By S. J. Crispin, M.I.Struct.E., L.R.I.B.A.

Soil Stabilisation in Fine Materials

The purpose of this paper is to give a description of the practical application of different methods of soil stabilisation in fine materials for the construction of road bases and the foundations of structures. The sequence of the sections follows the order of the operations in the job from the untouched site to the finished pavement. The observations made herein and the conclusions drawn from them are based on an intensive study of the working of a million cubic yards of excavation, a hundred and twenty thousand square yards of formation and fifty thousand cubic yards of soil stabilised with additives, the last two without the importation of hardcore or aggregates of any kind. The geographical situation of the site is immaterial, but the geological description of materials dealt with is most important. These descriptions could apply over large areas, since many alluvial strata, such as windblown and desert sands now allowed to go to waste, come within the particle sizes which may be stabilised by these methods.

Both the advantages and disadvantages of the methods suggested have been discussed, because it would be uninstructive only to describe the final method which has been found successful ; also it may be that some expedients which did not succeed in the particular cases observed may be of great use in other situations. Careful records and tests were kept of all the work done and the study of these, together with points of interest raised by visitors to the site, all contributed to the opinions expressed in the paper.

Survey

An accurate survey of the site is essential, both topographical for contours and natural features and geological for the nature of the ground. Correct levels throughout are necessary for fixing formation lines, balancing cut and fill and sectioning strata. Natural features should be plotted, for example, wooded areas, dry and wet areas, pasture, arable or waste land, rank and good grass land, scanty coppice and good timber. A careful study of these features is of assistance when making decisions based on the geological survey.

The geological survey is very important as the system of stabilisation is essentially the transformation of materials excavated on the site into a finished product with the minimum extraneous additions and its greatest economy is in avoiding the unnecessary transportation of material from one point to another. Expenditure on the survey will be found to be fully justified as the work progresses. The survey must be accurate ; it is useless to issue a general instruction to the contractor to take out trial pits, hoping for a good result from several diverse sources. The types of strata and their position are an element of design and all persons concerned in it should be thoroughly conversant with them. It should not be surmised that a hill consists of sand throughout because a sand pit shows on one side ; it is essential to make sure. This principle cannot be overemphasised, it applies equally to roads and to foundations.

The term soil is hereafter used in the engineering sense, that is the uncemented, uncrystalline portion of the ground between the agricultural soil and solid rock. Diagram I is a typical longitudinal section of soil worked in shallow levels and having diverse strata of varying usefulness in the work. The symbols in the diagram denote these strata, which are as follows :—topsoil, having a large organic content, which is of no structural value but is useful for resoiling ; clays which are not considered suitable for fill or stabilisation ; spoil which is a mixture of organic matter ; clay, etc., too irregular to serve any useful purpose ; fill, which is suitable for mechanical stabilisation, and sand, often with a small clay content but which is considered suitable for stabilisation with additives, within the limits of present knowledge. It is better to use simple terms on the site to describe the works than more abstruse geological terms. The thick vertical lines shown in the diagram denote boreholes taken out with a 6 in. auger at distances governed either by findings from the previous hole or by natural features. The excavated materials from the auger holes require careful observation and it is essential that they be laid out systematically to obtain a true picture of the strata. A hole made by a backacter gives an exposed face from which decisions can be made by visual inspection. In many cases such a machine may be available and although its use may seem drastic the opened-up cut gives a greater feeling of sureness of judgement than could be obtained by looking 12 feet down a 6 in. diameter borehole or even by inspecting its contents laid on the ground.

The thick horizontal line in diagram denotes the formation line and when this is plotted calculations of cut and fill, the positioning of spoil and other heaps and the estimation of the amount of stabilisable material available can be made.

As the sections show, in only two places, *A* and *B*, where masses of suitable sand occur above the formation line, is it economical to excavate with care, and if this system continues outside the working area it is better to widen the excavations at such points than to follow a thin sand layer as at point *C*. It would be sound policy to drop the level of formation between points marked *D* and *E* because after removing the four feet of topsoil the remainder of the excavation would supply suitable material. It is often more economical to drive into a hill than to avoid it. Detailed geological information is required to work a site to the best advantage. In the official survey of a certain site only three types of strata were mentioned in an area of approximately a thousand acres : Bagshot sand, Bracklesham beds and alluvium ; the first, which is the stabilisable material, was found at many points, but only in a few was it at an economical depth. A personal survey of the natural features is a great assistance, far too few designers and draughtsmen see the sites, with the result that unsuitable portions of the site are not recognised from the survey and therefore not shown on the plans as either to be avoided or to be drained.

Excavation

Diagram II. Excavation for soil stabilisation is a very important matter as careless excavation results in the loss of suitable material and the undue disturbance of

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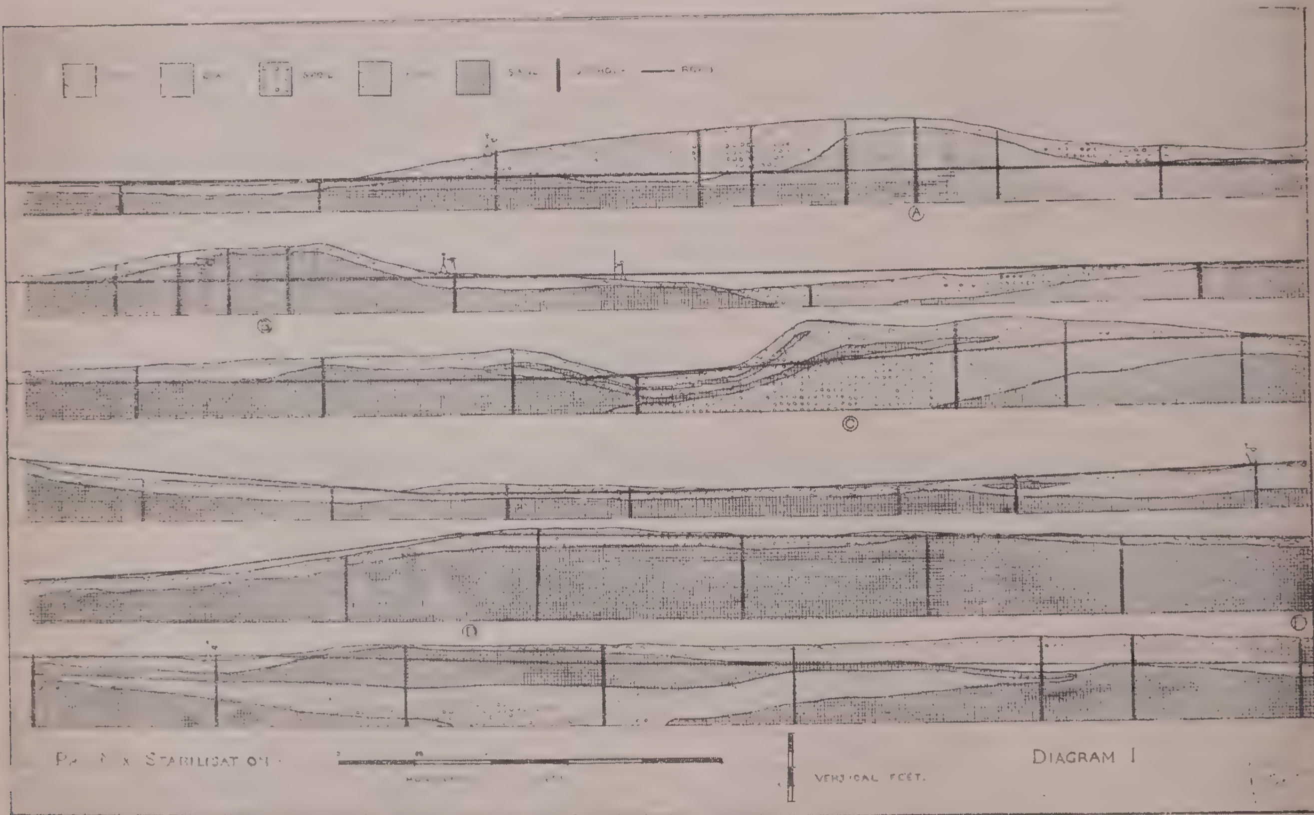
the formation. In order to make the greatest possible use of the existing materials, it is necessary to take out the excavations in layers and not merely remove the ground in a haphazard fashion. It is most important to preserve working material during excavation for the pre-mix method and to keep the working bed intact if mixing in place. Both supervisory staff and operatives should be trained to take an interest in the geological strata and the ability to recognise material texture by colour is soon acquired. The dark organic topsoil is easily recognised and its stripping a simple matter ; clays are recognised both by their colour and the effect on the soil left by the use of machines, some machines



cutting the clay smoothly and some leaving parallel cracks.

Working in vertical cuts through good and bad layers is a source of much waste on many sites. Questions of suitable and unsuitable fill or stabilisable sand are a continuous source of discussion on all jobs. The argument ranges from supporting the tendency to reject all site excavations and substitute off-site materials to the insistence that, with reasonable care and correct working, a large proportion of the site excavations can be utilised.

Clean sand is easily recognisable by simple field tests but sand with silt or clay content requires a laboratory test and comparison with an agreed standard of suitable material. Good sand ground is often covered by a different colour stratum, and during excavation this should be carefully observed and the contours of the useful material followed as they reveal it. Some beds of the Bagshot sand are intermittently interleaved with clays, but in the Bagshot sand proper depth is often only governed by ground water level.

Face Shovel. For general excavation work the ½-yard excavator (Dia. IIa) is very useful having as a face shovel a cutting height of 16 ft. 6 in. at a 45° boom angle. It is, however, not suitable when excavating overlaid horizontal stabilisable strata for, as the scoop rises, it mixes sand, clay, fill and topsoil into a useless mass of material from the point of view of stabilisation, providing an example of uneconomical methods of working.

Trenches. It is useful as a trencher or in a cul-de-sac but, as it stands on the future formation, care must be taken not unduly to disturb the latter. Where the excavator straddles a trench and cannot move backwards, it would be a great advantage if a side-acting excavator were available. This also applies when working near fences or trees or near the top of excavations. On superelevations, crossfalls and irregular ground, there is a great waste of material, unless the lorry and the machine are both correctly placed.

Skimmer. As a skimmer the ½-yard excavator (Dia. IIb) is a useful machine, as topsoil can be stripped clean or any strata taken out in layers, and comparatively slow working is compensated for by the obtaining of useful materials.

Back-Acter. As a drag shovel or back-acter (Dia. IIc), it serves a similar purpose in particular situations.

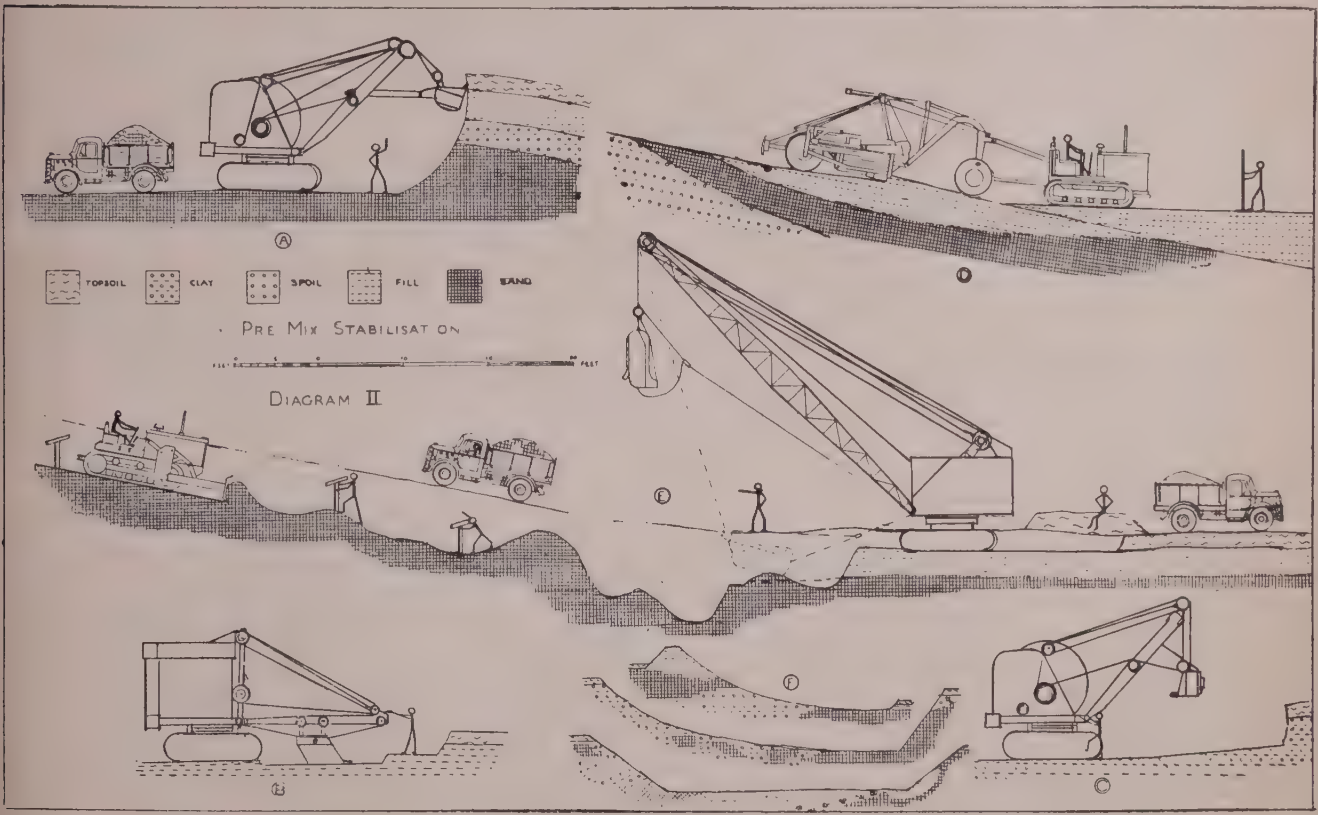
Scraper. Scrapers (Dia. IId) are useful for the removal of topsoil and overburden, taking large masses of useless material to spoil heaps, excavating and depositing fill, and also for moving stabilisable sand, when the latter is near the mixers. The scraper shown takes 12 to 15 cubic yards each trip and is considered economical up to a distance of 300 yards haul each way. It requires 25-30-ft. turning space. It is 33 ft. long, 10 ft. wide and has a cutting edge of 8 ft. 6 in. and 21 in. deep spread. Its weight is 9 tons inclusive of D.8 caterpillar tractor unit, and it can travel over seemingly impossible ground, climbing steep slopes and

forming cuttings and embankments a little wider than the machine.

Dragline. A dragline (Dia. IIE) with $1\frac{1}{2}$ cubic yard bucket does 600/700 cubic yards a day at 1 bucket a minute. For the same weight as the shovel type excavators, it has a greater reach, which is an advantage when it is desired to take out bad ground without undue disturbance, and also it has flexibility in action, a good driver being able to form parabolic cuts, crossfalls or banks as required. This machine can remove separate strata, but has a tendency to cause disturbance, which has to be made good afterwards. In loading, there is little waste of material, as the bucket is poised directly over the lorry.

Bulldozers are not always suitable for the type of work in which great care has to be taken to remove the soil without disturbance of the strata.

Sections. (Dia. IIF). The sections shown in the diagram are examples of varied strata forming the surface of cuts, all of which are required to be brought to a regular density during the progress of the work.



Drainage

Site drainage is of two kinds, the first consisting of ordinary surface water drainage including that from impervious surfaces and the diversion of existing streams and ditches ; the second being the drainage necessary to bring the sub-base to a moisture content suitable for compaction to a required density. Agricultural drains are used as primary collectors and are connected to watertight stoneware main drains taking water to outfalls. Impervious surfaces require drains at all their edges except where superelevations or crossfalls exist. In taking surface water from large areas into agricultural drains care must be exercised to ensure that water running over the lip of the impervious layer does not undercut into a softer substrata, in such cases a small kerb with definite outlets connected to the main drain is desirable.

The drainage necessary for compaction of the sub-base is governed by weather conditions and unforeseen natural water sources, and is difficult to design before the work is commenced, especially as regards which portions are to be permanent and which temporary. A provisional sum to be used as required is the most equitable way to deal with this matter and good preliminary site investigation can keep the unknown expenditure within reasonable limits.

Drains Diagram IIIA shows that it is economical to keep large drains to the width of an excavator bucket. Consolidation drains as shown at IIIB should be taken one foot below formation and filled with gravel rejects or rubble ; if an agricultural drain pipe be used it must lie on the bottom of the trench or the rejects under it will become the watercourse and the pipe useless. Drains under the pavement or the fill should be cut by hand to prevent disturbance of the formation and they should be as narrow as possible to prevent future scour. Drains in excavated portions (as shown at IIIC) where ground water is likely to break out at formation line should be arranged to drop the ground water level at least one foot below formation, in many cases this will allow the latter to be compacted by rolling. Any fill which is being used to replace poor strata must be consolidated as it is laid to prevent water being drawn from surrounding ground by a mass of fill at low density.

Erosion. Whether there be any erosion in rubble-filled trenches in fine strata is a debatable point ; generally the water flow is slow and shallow in the trench and pipes in the bottom corners of it help to overcome any such tendency. Silt which is often attributed to this

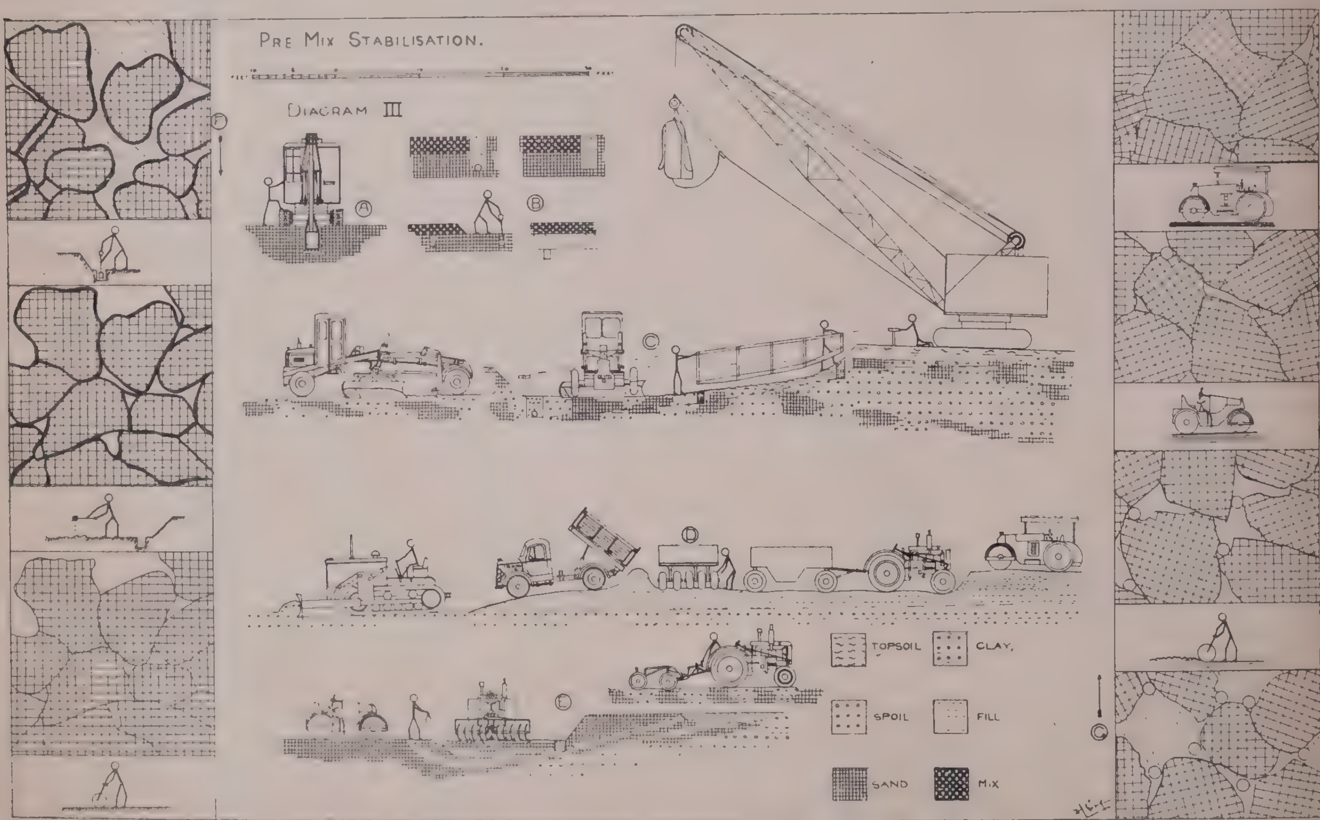
action is much more likely to accumulate from dirty surroundings, freshly formed runnels or badly formed banks adjoining the trenches. The system of cutting excavations with vertical sides along the outer sides of the drain and taking out the batters later is also another cause of silting up the drain. The gravel-studded sides found when the latter type of trench is opened up are proof that their erosion is negligible.

Formation

The greatest problem on a site and the one largely governing the programme is not soil stabilisation by additives, which will be dealt with later, but the mechanical stabilisation of the formation or foundation to carry the pavement or slab. When such a body has the property of resistance to lateral displacement it is said to be mechanically stable.

As the particle size gets less stabilisation becomes more difficult ; the particles under consideration in this paper practically all pass through a 52 sieve and include different kinds of sands, silt, clays and many combinations of these interleaved with peat, coal and fossiliferous deposits. The footprint left by the dinosaur in soft ground for the geologist millions cf years later, the water-bound macadam of 50 years ago which wore away by abrasion and not mechanical collapse and the more scientific soil stabilisation of to-day are all examples of the same process. The formation must always be brought to a definite density, governed by tests on the various strata. It presents many difficulties, one of the greatest being the tendency of certain soils, generally those of a sand-clay mixture, to become plastic and almost liquid when a load passes over them. Although such a mixture will allow of being walked on when exposed, yet, after a few passes of a roller in order to bring it to the required density, it will not carry even a small weight without lateral movement. This liquefaction is accentuated by vibrating rollers. There are many reasons to account for it, among them the pumping of water from adjoining strata, over compaction, excess moisture content and grain shape. This action and the methods to overcome it require very careful study in working fine materials.

The whole success of mechanical stabilisation depends primarily on the control of the moisture content. Text-books give much information about the addition of water but little about site methods of decreasing the moisture content. This is the predominant problem on the site which always seems to recur under different circumstances. The traditionalists try to overcome the trouble by putting down cinders and hardcore, but such action is against the principles of soil stabilisation and is out of date. The load bearing base should be formed from materials on the site, and it is uneconomical to use cinders, brick and hardcore which are not available everywhere and require transport. The formation of the sub-base from materials on the site is of use in all fields of engineering and is not confined to work where stabilisation by additives is to be used, the saving of hardcore in the sub-base is as important as the saving of aggregate in the pavement.



Plant. The formation line can be set by boning in a wood profile shaped to the required section or by boning in rods alone, Dia. IIIc. These are simple methods but they have the disadvantage of making it likely that separate parts of the plant may become scattered over the site, dislodged, or lost. A better method is a well-constructed profile resting on fixed side rails from which all measurements are taken, especially when constructing cross falls or superelevated sections. The dragline is useful in curved formations but must be worked with care and skill, acting as a gentle skimmer rather than a drastic digger. Another very useful machine for formation is the Aveling-Austin 99 H Grader, as on left of Dia. IIIc, weighing 10 tons

and having an overall length of 24 ft. 3 in., a width of approximately 8 ft. and a turning radius of 31 ft. The blade, which is 13 ft. long, has a universal movement from 45 degrees on one side to the same on the other. The many levers in its cab appear somewhat intimidating but a good operator has wide opportunities to increase his range of action.

REPLACEMENT. Ground that cannot be brought to the required density and stability should be excavated to a depth of about 2 feet and replaced by suitable fill, gradually increasing in density from the bottom upwards, as shown in Dia. IIID.

The various operations described in the following paragraphs, are shown in Dia. IIIG, and have been traced from micro-projector slides. The sand grains are shown cross hatched, water is indicated by diagonally dotted lines and the small circles denote trapped air.

The bottom layer of refill should be rolled with a light roller until it shows signs of cracking or getting wetter, with the result that the particles have come closer together, some water has been forced out and part of the entrapped air removed. The next 6 in. layer should be rolled with a heavier roller which repeats the former operation and a greater density is obtained. This is continued in 6 in. layers until the formation level is reached. It is to be noted that a replacement two feet deep usually gives the required density at the top ; this is a better method to adopt than taking out ground to great depths to look for a stable base, ; the minimum disturbance of the natural ground is a basic principle.

Unsuitable material can be removed by a bulldozer or similar mechanical plant, Dia. IIID, and suitable fill compacted by a wobbly wheel roller consisting of a sand box on 10 wheels rolling slightly out of the vertical and all adjustable for weight by altering the load in the box. In some cases a system of small drains at 12-foot intervals filled with pea gravel or small rejects may be used for the primary consolidation of the natural ground but this action depends on the percolation of the water by gravitation only and is a very slow process.

AERATION. In very wet formations and those in which, although the required density has been obtained, the ground forms a plastic material which goes out of shape under any applied force the method shown in Dia. IIIF should be adopted. Sand grains are indicated by cross hatching, water by diagonal dotted lines and the thick line around the particles represent water attached to the particles by capillary attraction. If open trenches be dug at intervals the free water runs off by gravity and allows the particles to consolidate but they still retain the water held by capillary attraction. The mass remains plastic as the water is sufficient to lubricate the particles which easily slide over each other and prevent stability. If dry sand is placed between two microscope slides and damped very slightly it will demonstrate the intensity of this lubrication. The surface water may be evaporated by forking, disc harrowing or any other methods that will expose as much particle surface as possible to the sun or to a drying wind. The proper moisture content that will allow the mass to attain a stable condition when rolled may be judged either by experience or by taking trials at intervals.

SOFT SPOTS. Soft spots are isolated areas in the formation which are not capable of being brought to a required density or of being mechanically stabilised.

A geological formation that would cause such a spot is illustrated in Dia. IIIE, where a sand-clay strata, fed from an adjoining pool, is exposed at formation level. The removal of the source of the water, agricultural drainage and aeration all will help to solve this problem. If the water cannot be completely removed it is only necessary to obtain the required stability during the time of construction and allow the ground to revert to its natural condition when sealed by the finished overlay.

Examples may be found of many heavily loaded areas which rest on a confined layer of low consistency.

Other soft spots are likely to occur where the formation line cuts the original ground line having fill on one side and excavation on the other, the centre coinciding with the original ground level. The fill should be consolidated in thin layers and the surfaces exposed by excavation compacted by rolling but the centre may yet remain or become soft and insecure. The undue use of formation by constructional traffic without providing protective carpets is another cause of soft spots. Although the density tests in such areas may be up to requirements and on their results alone work would continue, these tests do not always prove that the surface can be worked or consolidated, or both. Hard portions sometimes give lower density and higher moisture than apparent soft spots which are shown by simple pressure tests or visibility alone, to require further treatment. On the other hand, places that are exactly similar in appearance and give equal results for tests may require different treatments.

Either the provision of small drains, excavation to a moderate depth and replacement by suitable fill or aeration as described or some combination of these three will usually overcome these difficulties. The stabilisation of such places by the direct insertion of additives is not generally successful. Small trenches remain dry for a considerable time when free water is not the trouble and the drying of the surrounding ground may be accelerated by aeration.

SURFACES. Surfaces once brought to the required density do not always remain unaffected ; depressions, footprints, areas bounded by superior layers and other areas which hold water, are serious faults ; the water should be drained off, the top surface removed and replaced with consolidated material at once as many soft spots originate from places such as these.

Mixing

Two methods of mixing stabilised soil are in use, the "mix-in-place" and the "pre-mix," and on every job an early decision must be made as to which method it is better to adopt.

The mix-in place method is suitable where a naturally stabilisable material occurs at formation level and mixing with cement or other additives can take place without the removal of the basic material from its natural position. The work may be done with mobile mechanical plant or even with hand tools where comparatively light construction is required. Many of its advantages are lost when varying strata are being dealt with.

The pre-mix method is most suitable in places where large quantities of the stabilisable material only occur in certain portions of the site and can be obtained either from excavations in the working area or from nearby borrow pits. The material may then be taken to a large central mixing plant or to a number of smaller mixing plants spread over the site. In some cases a mobile pre-mixing plant may be an advantage. The pre-mix method allows better control of the mix and allows an unlimited thickness of pavement when large loads have to be carried or the stabilised soil is used as

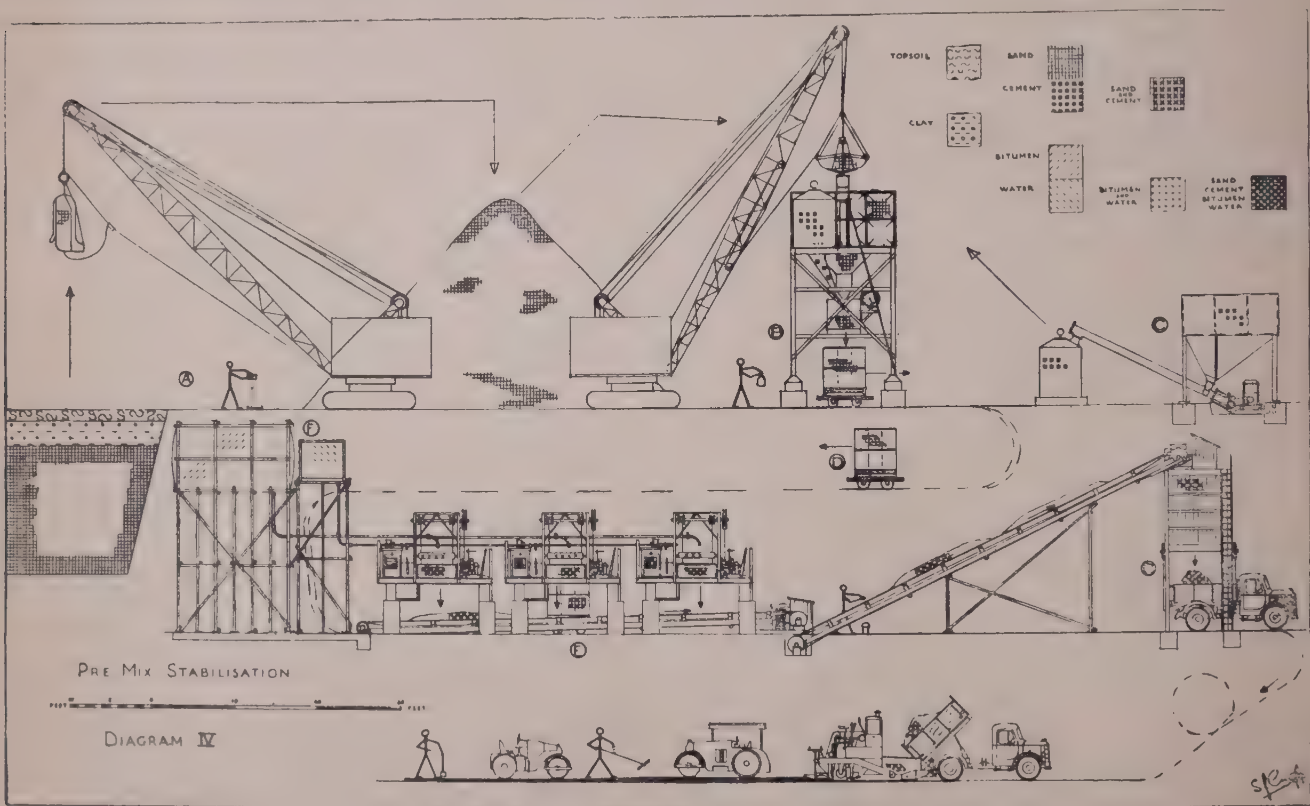
a structural material. Work on embankments, or in cuttings formed of or in unsuitable materials, or on foundations in varying strata is possible by this method.

Pre-Mixing Plant. Diagram IV is the lay-out of a large pre-mixing plant and shows the sequence of operations from the excavation of suitable materials to the finished pavement. The mix consists of sand with 10 per cent. cement, 2 per cent. bitumen emulsion and water. The symbols in the right-hand top corner of the diagram are marked on parts of the plant to show its contents and have been superimposed to show the sequence of operations.

In Dia. IVA the top soil and overburden are shown removed, the suitable sand under being raised by a grab

These loaded trucks run on jubilee tracks to the scoops of the mixers, Dia. IVE. The arrangement of these tracks must be carefully considered or traffic congestion between the weigh-batcher and the mixers can waste much time. A continuous circuit is preferable to an arrangement requiring shunting or reversal of trucks and the time taken in weigh-batching materials and the mixing time per batch must be carefully considered ; it is also preferable to provide enclosed feeding arrangements between the various sections of the plant to prevent loss of cement, etc., in inclement weather and the spoiling of a well-designed mix.

Bitumen emulsion and water tanks, Dia. IVF, are placed to feed the mixers by gravity through a pipe line with draw-offs as required. The paddle type mixers are



and deposited in conical heaps near the weigh-batcher. This method, reminiscent of hay-making, helps to remove excess moisture content and the heaps should be covered by a light roof or tarpaulins as a protection against rain. Moisture content tests taken from these heaps govern the amount of water, if any, to be added later.

Dia. IVC shows cement being delivered in bulk and fed into Durham Containers by a worm action and raised to the platform alongside the sand bin on the weigh-batcher, Dia. IVB. The sand bin has a tapered bottom, glass lined sides and a small endless belt under the outlet at the bottom ; these precautions are often necessary because of the tendency of fine sand to form a consolidated mass in the bin and restrict or stop its free flow. Prodding and the application of poker or form vibrators tend to increase the consolidation and such apparatus as revolving paddles require too much power to act efficiently.

Sand and cement flow is controlled from a platform and after weighing passes to a three-compartment truck, Dia. IVD under it. Each compartment holds enough sand and cement for one mixer batch.

raised on concrete legs to feed from chutes under the mixing pan directly on to the belt conveyor. Above the mixing pan is a tipping trough, fitted with a depth stud to measure the amount of bitumen emulsion and a hand or other measure for water. Careful tipping of the trough is essential to ensure equal dispersal of liquids throughout the mix ; poor operation is shown very clearly by large bitumen flakes in the mix as delivered to the belt conveyor, and by its varying colour this is a very suitable point for close inspection or for taking samples.

A batch consists of about 500 lb. of sand, 50 lb. of cement, 10 lb. of bitumen emulsion and water to give a 10 to 12 per cent. moisture content. The sand and cement should be mixed for 30 seconds, then the bitumen and water are added, and all mixed together for another 90 seconds. The batch gives from 5 to 6 cubic feet of finished pavement. The mixes are comparatively dry, which can be appreciated by observing the slope of the belt conveyor taking the mixed materials to the top of the wet bin, Dia. IVG without segregation. Mixes should mould in the hand without showing moisture on

the surface and can be stored for periods of up to one hour.

From the wet bin the mix is transported to the laying point and spread, either by hand or mechanically. This process is dealt with more fully in a further section of the paper.

Laying

PAVEMENTS. Examples of pavements are shown in Diagram V.*A*. In A1 a heavy track is shown, designed to carry 120 ton loads travelling at 60 m.p.h. The pavement consists of 14 in. of stabilised soil laid in three thicknesses, because 6 in. is the maximum depth that can be compacted to the required density throughout without great difficulty. The stabilised soil is then overlaid with a 10 in. carpet of asphalte and tar macadam between precast concrete kerbs. A test section of track showed no faults or failure when opened up after the required load had passed over 5,000 times. At A2 is shown a road with 6 in. of stabilised soil and 2 in. of macadam which will carry heavy constructional and ordinary road traffic. At A3 a side road is shown with the same base as A2 but overlaid with tar and chippings. This may be used for ordinary motor vehicle and agricultural traffic, but its resistance to abrasion depends to a great extent on the cover being kept in good repair. A4 shows a drive of 4 in. of stabilised soil left uncovered. Such a drive constructed for experimental purposes carried traffic for a period of nearly two years without structural failure, but this is uneconomic. In all cases a protective coat is advocated and all exposed edges should be painted in tar or otherwise covered.

SYMBOLS. Directly under each example of laying shown in Diagram V, geological symbols are drawn denoting the ground for which it is most suitable and the containers of plant are also marked with symbols denoting their contents. The geometrical symbols at the bottom of the lines show the action in the materials upon which the plant is operating.

DISTRIBUTION. When a central mixing plant is used the distribution of material requires careful consideration, especially in a formation which would be adversely affected by traffic. In the wet season traffic should not be allowed over the formation after the face is exposed. It is possible to leave one foot of ground above the final formation to act as a protection but this is not advisable as it leaves a soft wet mass of material which must be dealt with later and it is much better to lay temporary tracks.

In dry weather traffic on a fine-grained formation leaves a powdery layer which requires added water for compaction or it will draw water from the mix when it is laid and thus decrease the optimum moisture content, making compaction of the pavement difficult or impossible. Mixing plant should be placed to take advantage of all existing roads and where none exist, service roads should be constructed to connect the plant to the laying points. To omit these roads is a false economy which will result in idle mixers and lose of time because of difficulties of access.

METHODS OF LAYING. It is essential to realise that in stabilising fine materials it is much easier to do it correctly than incorrectly. If the moisture content be incorrect it will not compact to density and will tend to crack; if the sub-grade be poor the stabilised soil as laid will follow any movements in it. It cannot be placed in position more easily by adding a little more water, as is too frequently done with concrete, stabilised soil cannot be laid with greater facility by rendering it soft and thus reducing its strength.

HAND LAYING. Diagram VB shows the first method of laying, the uncompacted mix being delivered in 5-ton tipping lorries, deposited into heaps spaced at intervals according to the thickness of pavement required, this operation should be directed by one person only. These heaps should be levelled out by hand or by a suitable grader to a thickness of approximately 40 per cent. more than the required finished depth and then rolled with about 10 passes of a 3 ton roller. It is useless to endeavour to speed up compaction by vibration or a heavy roller. Care must be taken to ensure that articles like half-brick thickness gauges are not left in the newly laid mix before the roller passes as these will cause a corrugated surface when rolling commences. The geological strip under the diagram of this method of laying shows that it is suitable for all kinds of ground which have been pre-compacted to the required density.

Geometrical symbols show the material passing from an uneven condition to a regular form which is afterwards compacted to the required density.

MECHANICAL LAYING. In the second method shown in Diagram VC the stabilised soil is delivered by 5-ton tipper lorries as before in its uneven state and tipped into a Barber-Greene Spreader as used on general road work, which lays a strip adjustable in width between 8 and 12 ft. at a thickness varying up to 6 in. The adjustable screed allows of laying either to camber or crossfall but as a general rule with stabilised soil the formation has been set to the true levels in order that the machine may lay an even thickness throughout. The accurate setting of the formation and base courses is most important where several layers are being placed and very small tolerances are required in the finished surface. The spreader is self-propelled and tamps the material as it lays it, but rolling is required afterwards to attain the required compaction. The spreader machine shown in VC is working in the direction of the arrow, a loaded lorry backs against the roller at the end of the spreader, is taken out of gear and moves with it and tips its load into the hopper, which is large enough to allow working between lorry loads. From the platform at the back the screeds may be adjusted by hand and the depth of the material being laid measured with a steel rule or gauge. There is a tendency when laying such fine material for the machine to leave a slight wave or corrugation after it has passed, which is more noticeable if the machine stops and restarts, or if there are any irregularities in the level or varying conditions of density in the formation underneath. These corrugations if not ironed out tend to increase in size with each succeeding layer.

Before rolling commences the surface should be tested with a straight edge for inequalities and high spots raked off and low spots filled. This is only a superficial treatment, and is especially necessary in hand laying. It should not be considered as peculiar to stabilised soil, since most pavements require a similar treatment. Experience is required in judging whether the thickness of the unrolled material is sufficient to obtain correct density and the required final thickness, as errors cannot be rectified by the addition of thin layers after the initial set has commenced. This material has the advantage (by reason of its dry consistency and the speed with which it is rolled into position) of allowing men to walk on it and rake it immediately it is laid, without ill effects. Another advantage lies in its low moisture content which permits laying at lower tem-

perature than for ordinary concrete. While good curing obviates setting cracks, if the surface is likely to remain exposed for any length of time a tacking coat of tar and grit should be used; care should, however, be taken that this does not retard the initial set.

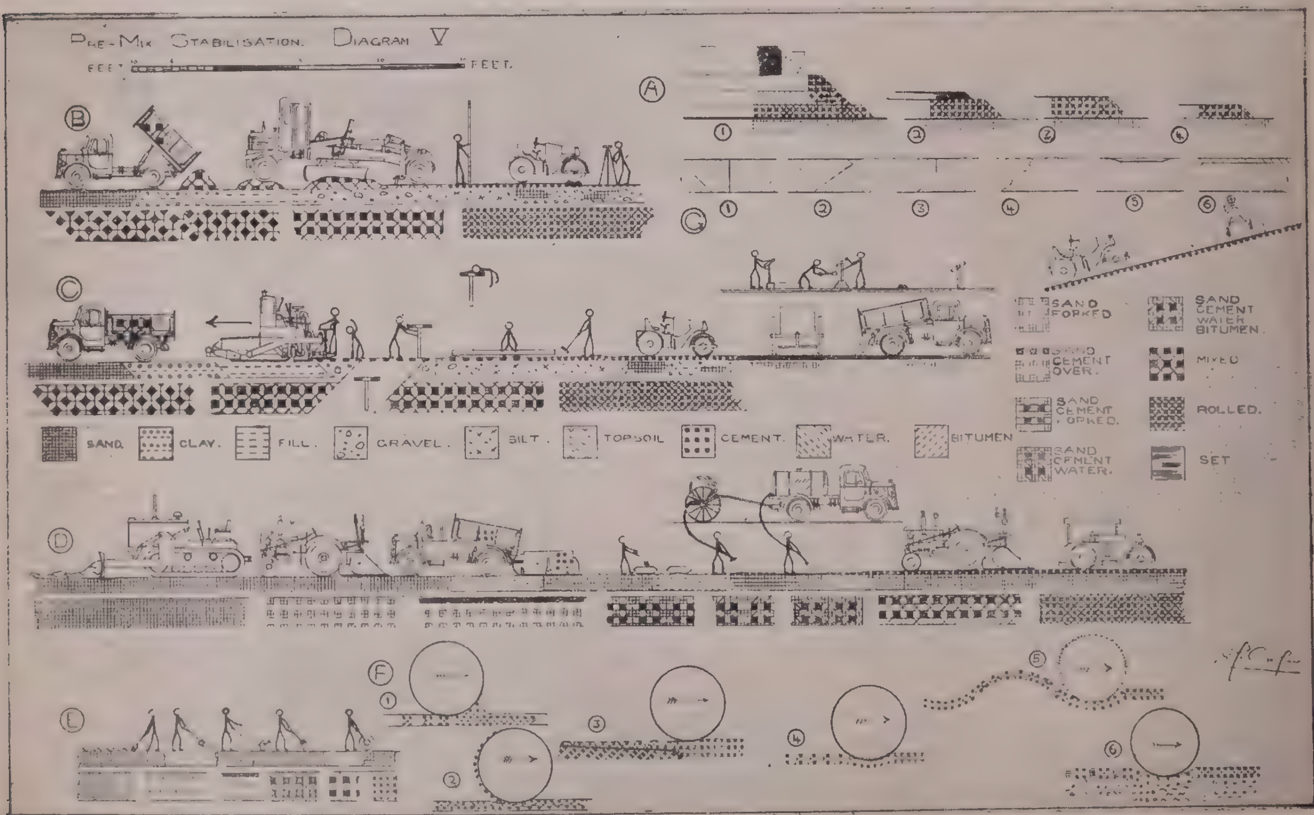
The method of mechanical laying can also be used on all kinds of strata provided they can be mechanically stabilised and its sequence of operations is similar to the foregoing. On steep grades, cross falls and super-elevations two light rollers working across each other are a great advantage.

MIX-IN-PLACE. Diagram VD shows one mix-in-place method which is suitable in regular geological strata, for use in laying material to a thickness of about 6 in. It can be used to take normal traffic, or as a substitute for hardcore in soil of organic or clay content. The first operation is to take off the turf either by hand or with a light bulldozer. The next is to fork over the ground with a Seaman Self-propelled Pulvimixer, consisting of a tractor in front with an enclosed rotar at the back, or other suitable plant. The Pulvimixer forks or stirs the ground to a depth of about 6 in. forming a suitable bed to take the cement. About $\frac{3}{4}$ in. thickness of cement is then laid evenly by a mechanical spreader attached to a lorry; alternatively, this operation can be done by hand. During further passes of the Pulvimixer the cement and soil are thoroughly mixed with the addition of bitumen and water as required. Even distribution of the added materials is very important. The geological symbols in the diagram show that this method is suitable in regular strata or on selected fill; the geometrical symbols under that all operations described take place in the ground itself.

PATHS. Diagram VE is intended to show that mechanical plant is not required in order to do a simple job for carrying light loads. For example, all the equipment required to stabilise a garden path is suitable material for the mix and ordinary garden tools, such as spade, fork, watercan and roller.

EFFECTS OF ROLLING. In diagram VF figures 1 to 6 show the effects on the mix when it is laid and rolled in difficult conditions, usually governed by moisture content.

F.1. In this the mix is too dry, it has formed a ridge in front of the roller and will not compact. Although this is rare in this climate it can easily happen when the mix has been allowed to stand too long in a hot sun or drying wind, especially on slopes which face south.



F.2. If, as in this diagram, the mix is very wet it will stick to the roller and pieces will drop off tending to cause an irregular surface. Raking and aeration will be of assistance in remedying this condition.

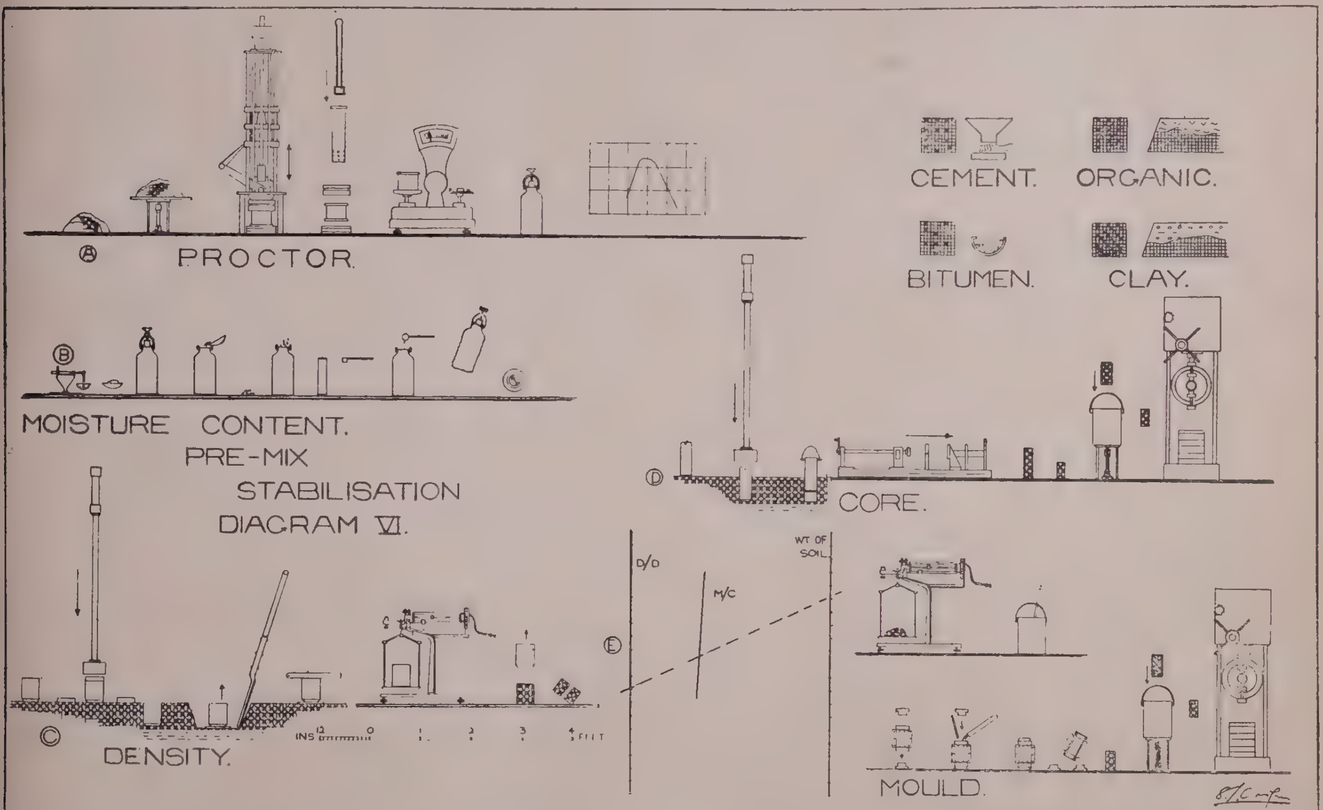
F.3. Thin layers often overlap at the joints of strips. This diagram shows that such overlaps also attempts to rectify incorrect levels with patches that are much less than 2 in. thick, fail because all thin layers or patches crack and break under the roller.

F.4. This represents a mix that is too soft, forming a depression under the roller and often causing the surface to become corrugated. This softness is often due to over wet spots in the formation below.

F.5. This diagram shows how a vibrating roller, by working itself into the mix, causes depressions and corrugated surfaces.

F.6. The diagram shows how excess moisture content in the mix or formation tends to form a springy surface when rolled making compaction impossible. This is similar to the action described under " Formation " but in this case is more difficult to rectify.

JOINTS. Stabilised soil is often laid without expansion joints and care must therefore be taken that contraction cracks, which must of necessity occur, do not coincide with the construction joints between laying strips. Diagram VG shows examples of good and bad jointing. Square edged joints as at G1. should be avoided; diagonal joints as at G2. form feather edged strips at the surface level which tend to break off when rolled; the half vertical and half diagonal joint as at G3. is good but difficult to obtain in practice. Generally for stopping at night or lunch hours a diagonal joint at about 60 degrees to the cross-section is good practice. When fairly continuous laying can be arranged, a rough joint as at G4. will combine with the next strip if laid within approximately one hour. All water filled surfaces and thin layers as shown at 5 and 6 are sources of weakness in the finished surface.



Tests

Tests for stabilised soil are fully dealt with in text books, British Standard Specifications, etc. and only a few general site tests are dealt with in this paper.

PROCTOR TEST. Diagram VIA. The first test to be made on either the sub-base or on materials for mixing should be a compaction test, such as the Standard Compaction or Proctor Test, from which the maximum dry density of the materials is ascertained. This test is done by applying a known amount of work to standard volume samples of the material with gradually increasing moisture content. From the graph of these results is obtained the maximum dry density and the optimum moisture content. The work in this test is applied either by mechanical or hand worked apparatus.

MOISTURE CONTENT TEST. Diagram VIB. The moisture content of a material can be obtained in many ways. On some sites a balance and a frying pan constitute all the apparatus. The Speedy method shown consists of a set of scales to weigh 26 grammes of sand which is placed in a metal bottle, sometimes with the addition of a few dry ¼ in. stones in order to break up the damp lumps of material. Two standard spoonfuls of powdered calcium carbide are placed in the cap of the bottle and drop into the bottle when the cap is replaced; the bottle is then well shaken, and, as the pressure exerted by the gas formed in the bottle is relative to the moisture content, it may be read off the dial in the bottom.

FIELD DENSITY TESTS. Diagram VIC. Field density tests are made with a 5 in. deep cylinder, 4 in. internal diameter with a sharp bottom edge, and a protective ring at the top, which is driven into the ground by a rammer. The full cylinder is then removed by excavating a square hole around it and placing a flat plate underneath. The top ring is removed, the sand struck off with a straight edge to the top of the cylinder and the whole weighed. The moisture content is obtained and the dry density can be calculated and compared with the standard compaction test.

COMPRESSIVE STRENGTH TESTS. Compressive strength tests of the stabilised soil may be carried out in several ways. Diagram VID, shows the core cutting method in which an 8 in. long cylinder with an internal diameter of 2 in. having a sharp bottom edge is driven into the material to be tested when it has been laid and rolled or compacted to the required density. The full cylinder is removed, the core extruded and cut to leave a cylinder 4 in. long and 2 in. diameter. The cone is dipped in wax and kept in an even temperature for seven days and then tested in an unconfined compression testing apparatus. This is a very direct test and can only be applied to finer materials.

The second test is the moulded cylinder test shown in Diagram VIE. This is done by first obtaining the dry density and moisture content of the material and then by

calculation or the use of a prepared scale ascertaining the weight of the material required to fill a constant volume mould at the required density. The standard mould is filled by tamping the whole of this amount of material in three layers and replacing the caps under pressure, the mould is then divided, the test cylinder removed and dipped in wax for testing after the required period has elapsed.

COMPARISON OF TESTS. The core cutting test is a direct test on the material as laid. The moulded test requires a considerable amount of hand work in its preparation. The latter, however, does not give such a true result of the efficacy of the work done on the site as the former.

The following is a comparison of a number of tests showing the method used and the average results obtained, all on the same sort of material.

Method of Preparation	No. of Tests	Average strength lb. per sq. in.
1. Laboratory moulded and water cured	6	516
2. Field moulded and water cured	2	448
3. Laboratory moulded and air cured	50	391
4. Field moulded and air cured	11	363
5. Cored	63	336
6. Site laboratory moulded	396	496
7. Site field moulded ...	363	450
8. Site cored	336	335

It will be seen that the results tend to be higher as the amount of laboratory work on them increases so that wherever a positive test of materials is practicable it should always be used.

In conclusion it may be said that the following rules for exact working cannot be over-emphasised. The selection of plant must be governed by site conditions and the importance of reducing the disturbance of the natural ground to a minimum constantly borne in mind. The exact times of mixing and testing must be clearly stated in the specification and there should be a clause that all work must be left exposed until the results are known. It should be remembered that good site investigation at an early date will allow of more definite information being included in the specification; it should also be required that records be kept by which portions of the work may be easily related to test results.

The observations and opinions expressed in the paper have been formed during the progress of a large contract carried out for the Ministry of Works by Messrs. A. Monk and Co., Ltd., of London and Warrington. The writer would like to thank many persons connected with both of these for their help and co-operation and the information they have given him, while admitting that they will possibly not always be in agreement with the conclusions which have been drawn from their observations.

Messrs. Goffe Cooper, Messrs. Aveling Austin and others have also produced information with regard to plant.

Finally it is hoped that by following the general principles here stated some of the present difficulties may be overcome and soil stabilisation with or without additives may become more widely used.

Book Reviews

Design of Cylindrical Concrete Shell Roofs. (American Society of Civil Engineers. Manual No. 31. 177 pp., 9 in. × 6 in. $5.00.)

Engineers will welcome the appearance of this Manual, the result of more than four years' work by a Committee of the American Society of Civil Engineers under the chairmanship of C. S. Whitney.

The design of concrete cylindrical shell roofs according to the elastic theory is explained in relatively simple terms and tables are given by the use of which the forces in the more straightforward types of shell may be computed in very little time. The manner of using the tables is demonstrated by worked-out examples, one of which is worked through as far as the design of the reinforcement.

In order that more complex cases not covered by the tables may be solved, the formulæ which the designer will need in his analysis are conveniently listed and an example shows a method of setting out the computations.

The theoretical basis for the tables and design formulæ is given mainly in an Appendix. The analysis is new and contains simplifications appropriate to the shell proportions normally encountered; it remains to be examined critically. A key to the notation is usefully placed at the end of the Manual.

It should not be thought that this publication reduces the designing of shells to a matter of substitution in formulæ. Although the mathematics involved is simple, much engineering understanding is required. Nevertheless, the use of tables and the demonstration of the arrangement of the work do enable the engineer to solve a number of shells fairly quickly and in the result cannot do other than increase interest in shell construction and lead to an appreciation of the effects of the factors which influence their design.

F. G. T.

Forces in Framed Structures, by T. Lyle Morgan. (London: Spon. 1952.) 9 in. × 6 in. 215 pp. 25s.

This book deals with the analysis of simple or statically determinate framed structures at some length and perhaps it would have been better to have entitled it "Forces in Simple Framed Structures."

In his preface the author states that he considers the method of equilibrium of the joints to be the most powerful method available for the analysis of such structures. Other authorities and many practising engineers will prefer either graphical methods or the method of sections, but in actual fact no one method is the best in every case. When dealing with the method of tension coefficients, it would have been better if the equations had been set out in tabular form, especially when dealing with three-dimensional frames.

Chapter 7 is devoted to deflections and it is rather surprising that an author who is engaged in teaching has omitted the proof of the fundamental formula $\triangle = \frac{1}{E}\Sigma F_{AL} F_{UL} K$. It is still more surprising that this is used to find the displacements at every point, no mention being made of the Williot diagram or the use of Clerk Maxwell's Theorem of Reciprocal Deflections.

Apart from these criticisms, the text covers the subject-matter very fully.

There are a number of worked examples and also of questions with answers which will be very useful to students.

J. McH. Y.

Load Distribution in Prestressed Concrete Bridge Systems*

By P. B. Morice, B.Sc., Ph.D. and G. Little, M.Sc.

Summary

The problem of load distribution in a bridge is stated and various methods which have been suggested for its solution are mentioned. Methods which treat the problem through an elastically equivalent uniform system are studied in greater detail and the methods of practical calculation using distribution coefficients are explained.

The results of computational work of Guyon and Massonnet for no-torsion and torsion structures are given in the form of graphs which may be used for practical design and the method of calculation and the effects of torsion are discussed.

A description of tests of a number of interconnected prestressed beam systems is included and the results of these tests show good agreement with theoretical results. A point of interest is that the effects of distribution are virtually eliminated by a simple support and do not appear in the unloaded span of a continuous specimen. Examples of the method of calculation are given, two being taken from the experimental specimens and one of a more practical nature.

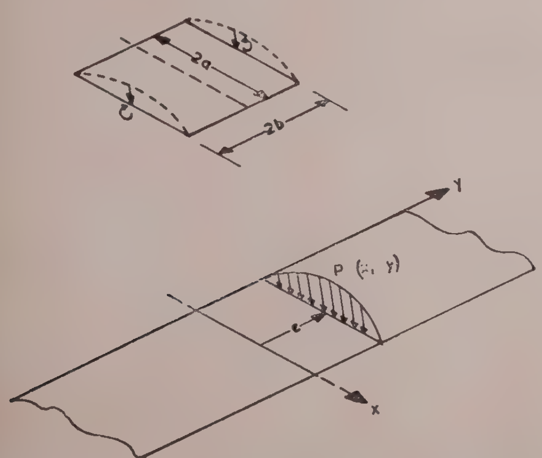


Fig. 1.—The two parts of the solution to the governing equation

1. Introduction

The purpose of this Paper is to demonstrate the use of a method of bridge design based upon a recently-developed method of analysis. The results of tests on prestressed concrete interconnected beams show that it predicts accurately the behaviour of such structures.

The problem which is involved is that of determining how a concentrated load or system of concentrated loads is distributed amongst the longitudinal beams of a bridge system for various degrees of transverse stiffness and torsional resistance. It is further of importance to the bridge designer to be able to determine an optimum value for the transverse stiffness and in particular, in the case of prestressed concrete, to determine the critical amount of the transverse prestress.

Although limits to the maximum laden weights of vehicles in general operation have been laid down by the Ministry of Transport, single abnormal indivisible loads of from 80 tons to 200 tons have not been uncommon. Certain routes known collectively as "The Short Term Grid" have been mapped for these loads. These are the routes on which bridges can be most economically strengthened to take such loads and are not necessarily the most direct. The ultimate aim is, naturally, for the routes to be in the form of the "Long Term Grid" in which they will follow the shortest trunk road from place to place. Thus, it is necessary for bridges on these routes to be designed for the standard loading but also checked for the abnormal loading of 180 tons.

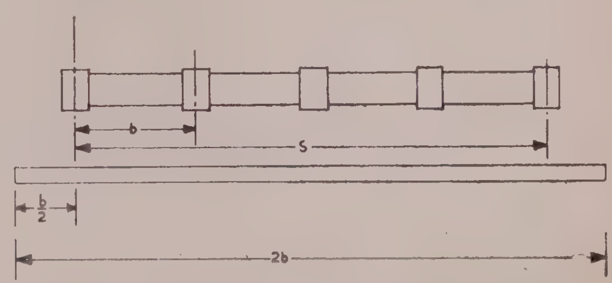


Fig. 2.—The effective width of a beam grillage

It is to be assumed that there is no other such load on the bridge at the same time and, hence, the amount of distribution to be allowed is an important factor.

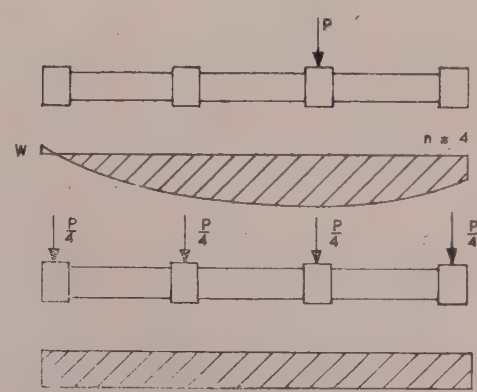


Fig. 3.—Uniformly distributed load and mean deflexion

Although the problem is not peculiar to prestressed concrete it is probably of more importance in prestressed concrete construction because reduced structural sections due to a more economical use of the material result in greater flexibility than in the case of reinforced concrete.

When starting the experimental investigation it was considered advisable to begin with the simplest structural forms and open beam grillages were selected for study.

**Paper to be read before a meeting of the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1, on Thursday, March 25th, 1954, at 6 p.m.*

It is appreciated that the bridge designer is more interested in a structure in which the decking slab contributes to the stiffness of the main and cross beams as in the cases of T beam and I beam and box beam forms. To this end the investigation is being continued beyond what is included in the present Paper which may be considered an introduction to the subject.

2. Review of Theories and the Distribution Coefficient Methods

The theories which exist for the description of the elastic behaviour of a system of interconnected beams fall into three distinct groups.

The first consists of theories which treat the structure in its actual form relating the behaviour of each member to the whole structure through a set of redundancy equations. The redundancy equations appear as linear simultaneous equations and for a structure of any degree of complexity will be large in number. The most satisfactory technique for dealing with such a problem is to make use of the relaxation process. Lazarides[1] has investigated this method of approach and has applied it successfully to beam grillages supported in a number of different ways including the bridge deck case.

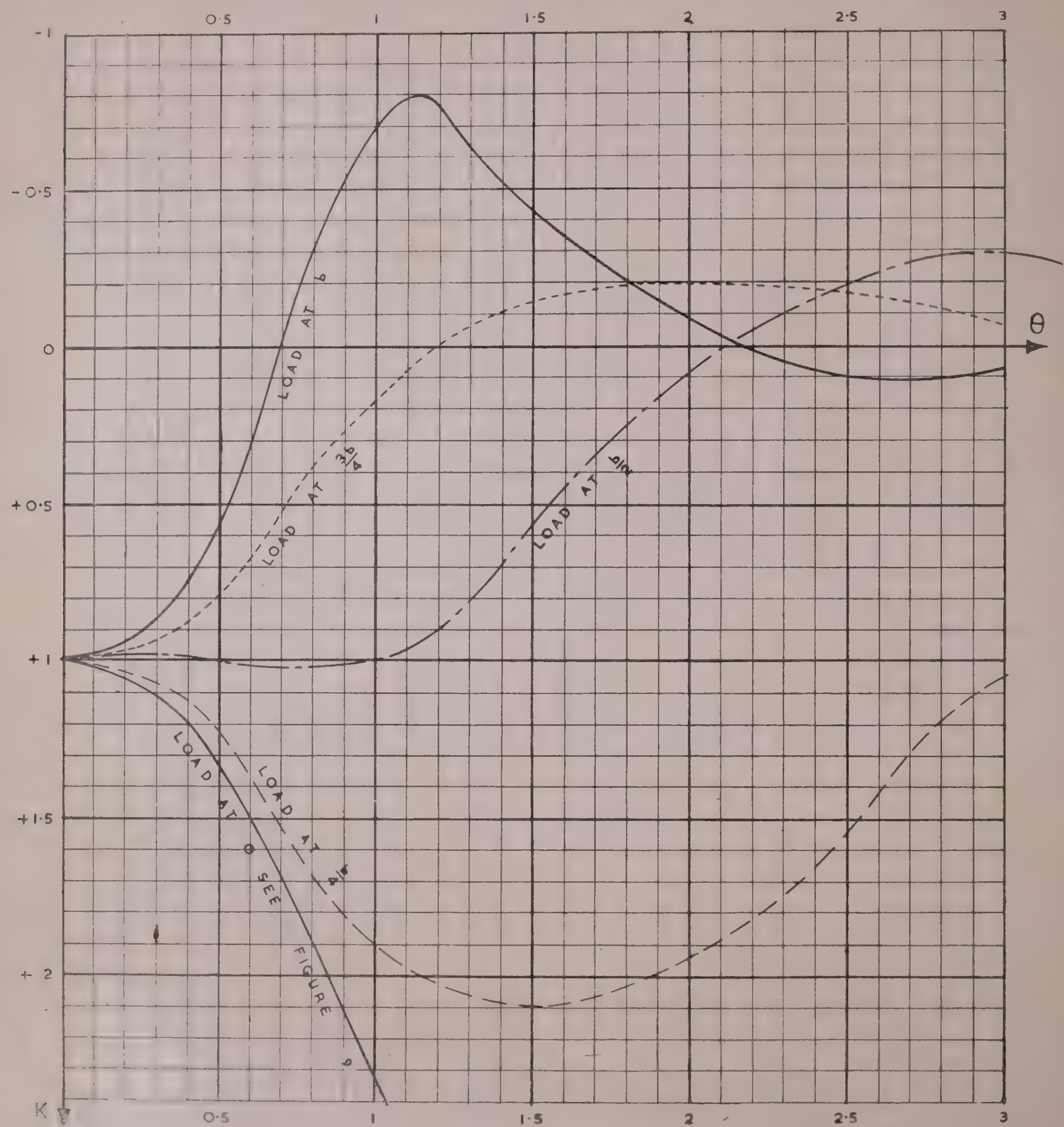


Fig. 4.—No-torsion distribution coefficients for a beam at $f = o$ (due to Guyon)

The second group of theories are those which replace the actual transverse connexion between the main beams by a more simple system of equivalent stiffness. The structural behaviour is then described by a set of simultaneous ordinary differential equations.

Pippard and de Waele[2] have used this method using uniformly distributed transverse stiffness as the simplification and shown that it may be used to provide a solution to the problem. The arithmetical work involved in any practical problem is very great and it is felt that it does not lend itself to design office use.

Hetenyi[3] has based his method upon the same idealization of the actual structure but has obtained consider-

able simplification by using sine series to represent both the load and deflexion of the structure in the direction of the main beams. This has the effect of eliminating the differential equations.

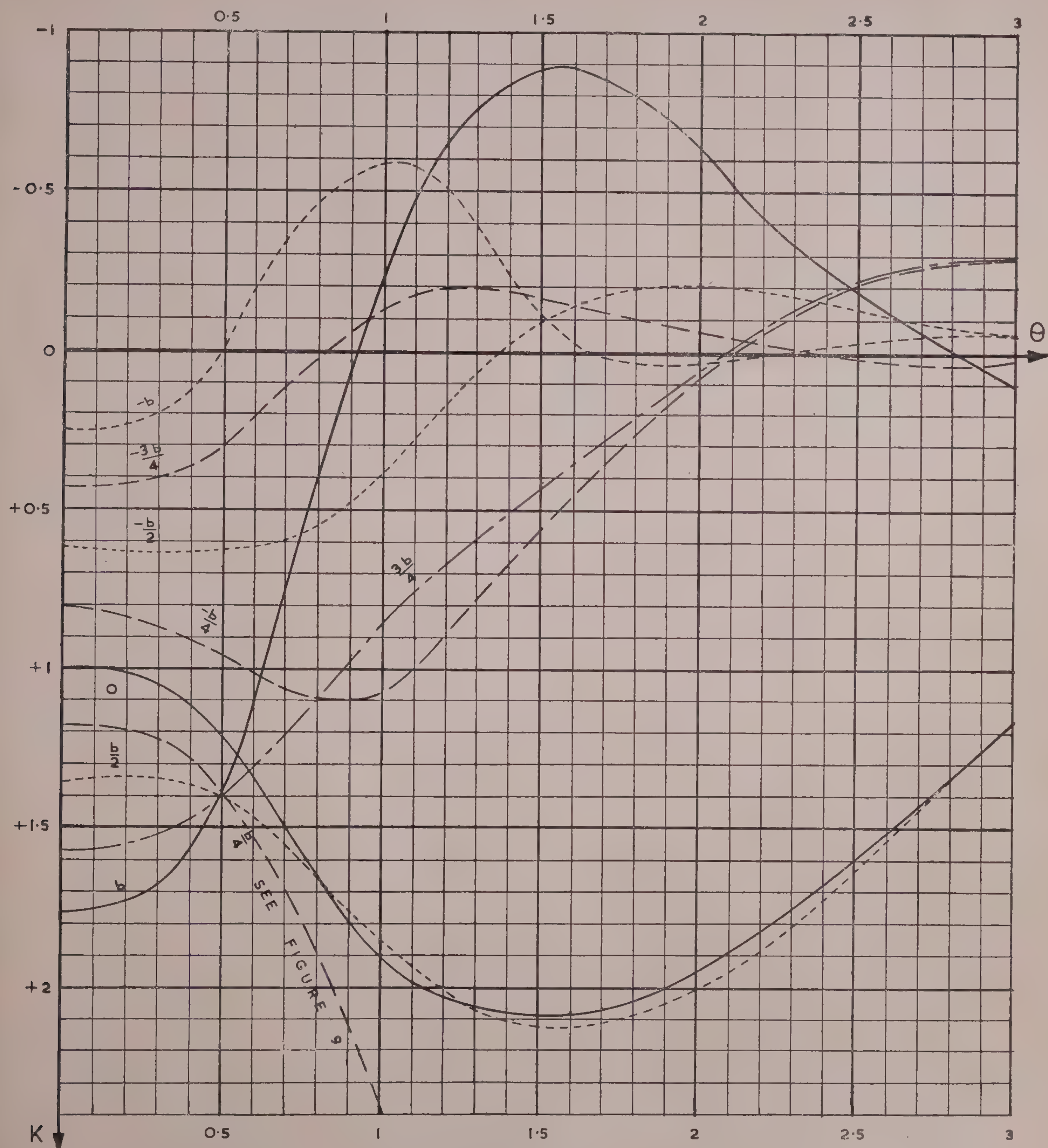


Fig. 5.—No-torsion distribution coefficients for a beam at $f = \frac{b}{4}$ (due to Guyon)

The method is one that can be considered as of practical value for design when the number of longitudinal, or main, beams does not exceed four.

Leonhardt[4] chooses a simplified cross-connexion in which all the cross beams are replaced by one central cross beam of equivalent stiffness. The effects of torsion are not included in these methods of analysis.

The third group of theories are those which reduce the actual system of discreet interconnected beams to an elastically equivalent system uniformly distributed in both directions.

The elastic equivalence between a general interconnected beam system and an orthotropic plate has been established by Timoshenko[5] who has shown that both are governed by the Lagrange equation

$$A \frac{\delta^4 w}{\delta x^4} + 2H \frac{\delta^4 w}{\delta x^2 \delta y^2} + B \frac{\delta^4 w}{\delta y^4} = P(x, y) \qquad (1)$$

where w is the deflexion of the system normal to the co-ordinate axes x and y and $P(x,y)$ is the surface loading function.

In the case of an orthotropic* plate A is the stiffness per unit width on a section $x =$ constant

$$A = \frac{E'xh^3}{12}$$ (h = plate thickness) and B is the corresponding stiffness on a section $y =$ constant

$$B = \frac{E'yh^3}{12}$$

The value of H is determined by the shear modulus and the Poisson's ratio and is written

$$H = \frac{h^3}{12}(E'' + 2G)$$

*The orthotropic stress-strain relations are written

$$\widehat{xx} = E'x\, e_{xx} + E'' e_{yy}$$
$$\widehat{yy} = E'y\, e_{yy} + E'' e_{xx}$$
$$\widehat{xy} = G e_{xy}$$

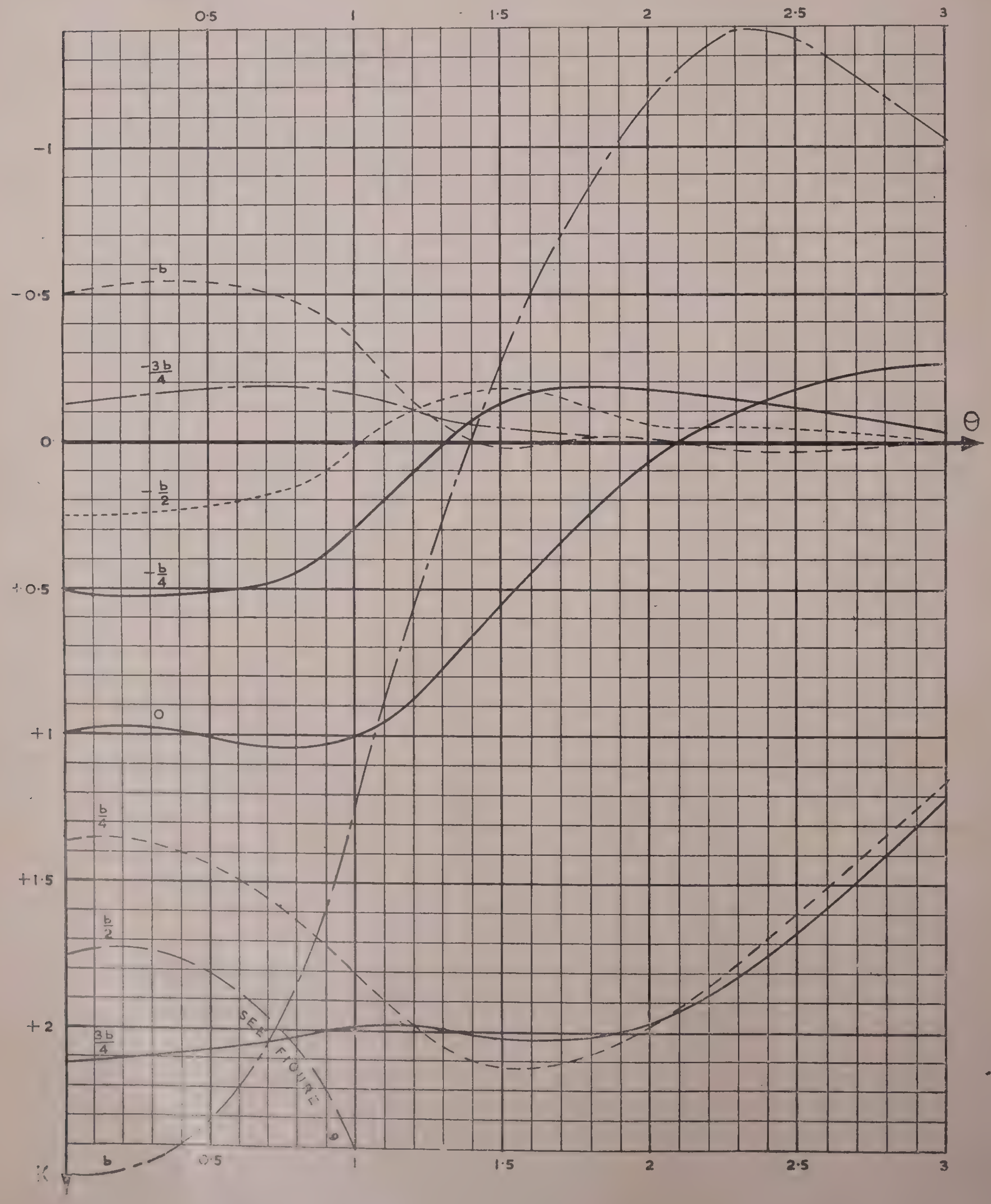


Fig. 6.—No-torsion distribution coefficients for a beam at $f = \frac{b}{2}$ (due to Guyon)

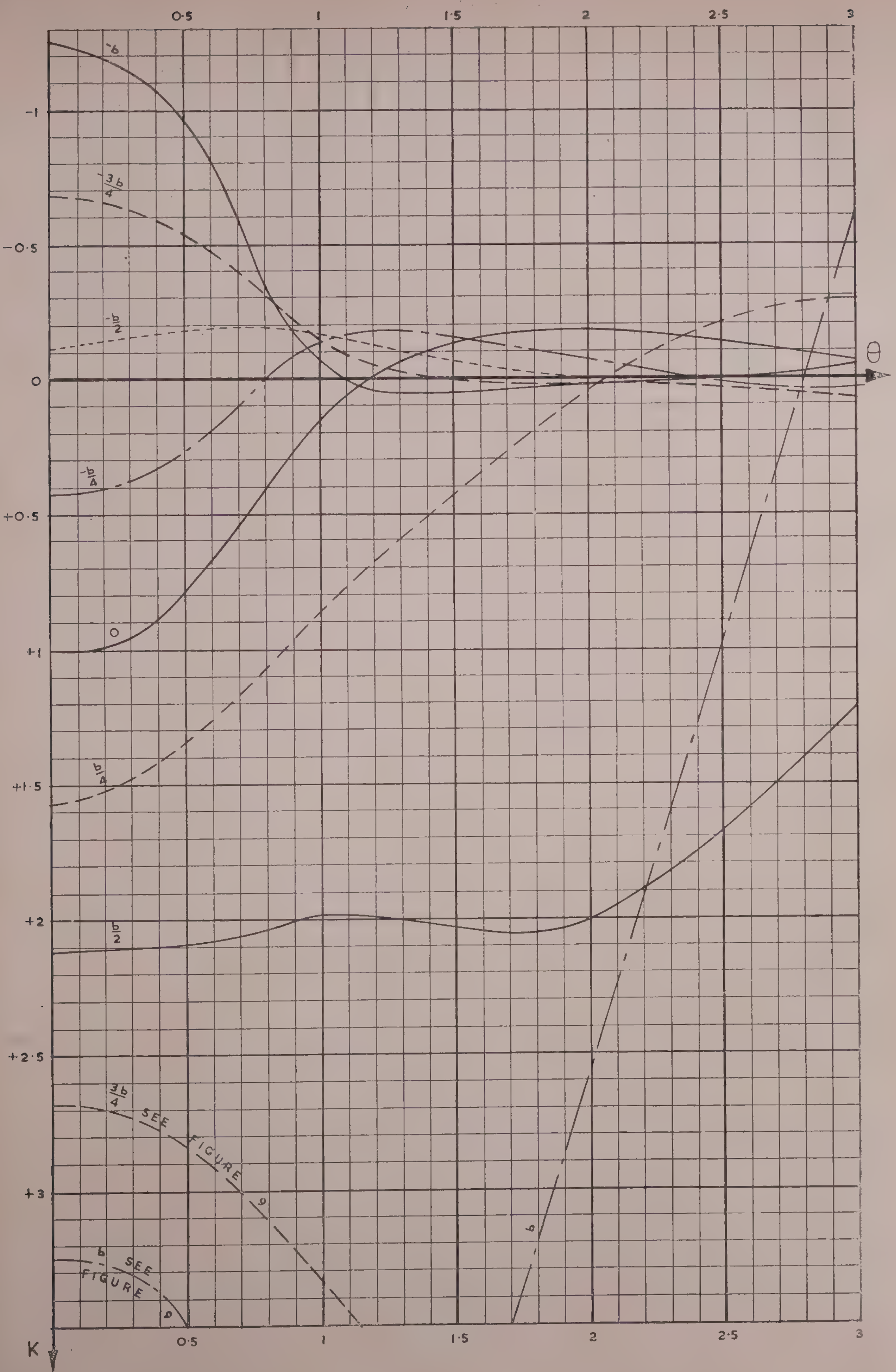


Fig. 7.—No-torsion distribution coefficients for a beam at $f = \frac{3b}{4}$ (due to Guyon)

In the case of a system of uniformly spaced interconnected beams the values of the constants A, B and H become

$$A = \frac{EI}{p} \qquad B = \frac{EJ}{q}$$

$$H = \frac{G}{2}\left(\frac{Io}{p} + \frac{Jo}{q}\right) = \alpha \sqrt{AB} \text{ (say)}$$

where EI is the stiffness of an x directional beam, EJ is the stiffness of a y directional beam and GI_0 and GJ_0 are

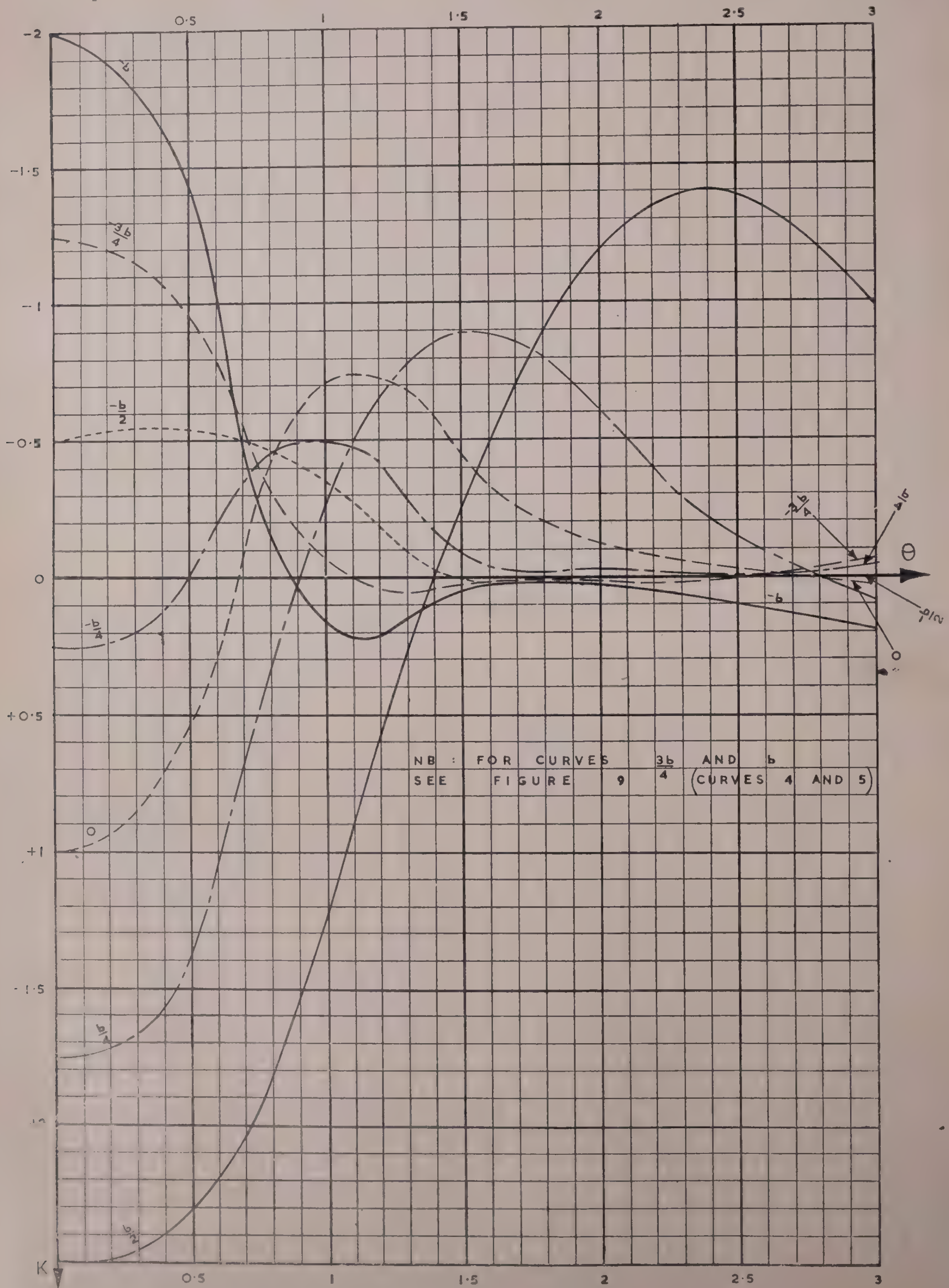


Fig. 8.—No-torsion distribution coefficients for a beam at $f = b$ (due to Guyon)

the torsional stiffness of the x directional and y directional beams, p and q are the beam spacings.

The replacement of the actual form of H by the function including α is convenient since α alone may be considered the torsion parameter with values between 0 and 1 for zero and complete torsion action.

Guyon[6] has made use of this elastic equivalence between plates and beam systems to use plate methods of analysis to study bridge beam systems. Guyon's analysis covers structures of zero torsional stiffness,

It is convenient to take it as the product of a function of x and a function of y and the Levy series form is adopted

$$w = \sum_{m=1}^{m=\infty} Y_m(y)\, Sin \frac{m\pi x}{2a}$$

where each term of the series satisfies the boundary conditions of simple support at $x = 0$ and $x = 2a$.

The harmonic form of the x dependent function reduces the partial differential equation (1) to an

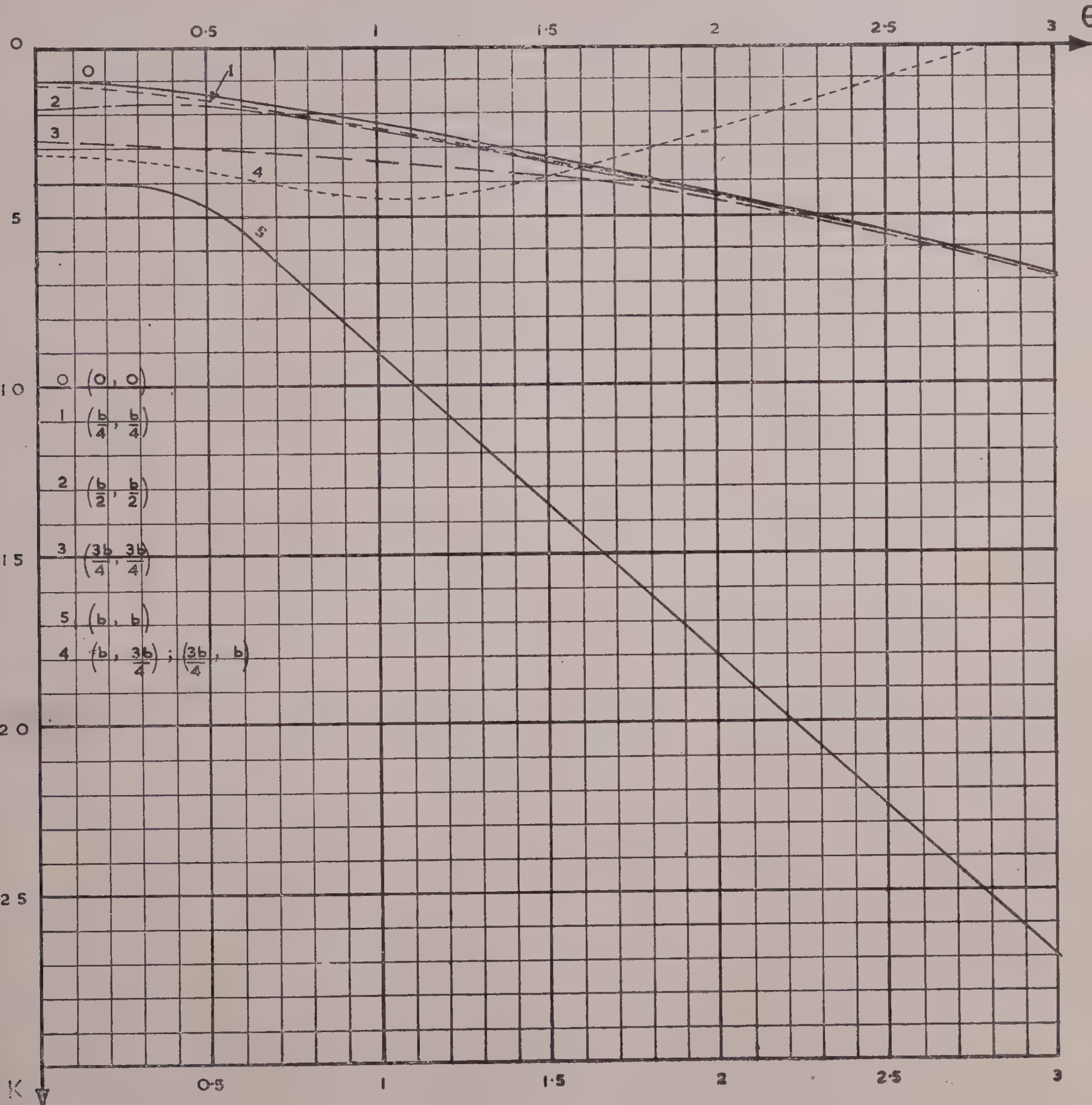


Fig. 9.—No-torsion distribution coefficients with large range

$\alpha = 0$, and he introduces a transverse stiffness parameter θ which has been retained in later studies.*

Massonnet[7] has extended Guyon's analysis to the general problem of a bridge system with $\alpha \neq 0$.

This is not the place to delve deeply into a discussion of the detailed mathematics of the theory but it is felt that a brief description of the main processes involved would be of interest.

The required solution of the governing equation (1) is an expression for the normal deflexion w.

The first step in the solution is to assume a form for the deflexion which is a function at both x and y.

*See page 8.

ordinary differential equation in Y with the first term only, $m = 1$, retained for practical calculation.

The solution of the differential equation consists of two parts called the particular integral and the complementary function. The particular integral represents the deflexion of an infinitely wide structure due to the imposed load $P\,(x,y)$. In this case $P\,(x,y)$ is taken to be a line load

$$P \sin \frac{\pi x}{2a}$$

at an arbitrary distance e from the axis $y = 0$ (Fig. 1).

The complementary function provides the deflexion pattern in a finite width structure due to boundary or

edge disturbances at the longitudinal edges. This solution includes four arbitrary constants which are found when the two degrees of freedom at each edge are specified. These freedoms are vertical displacement w and rotation about the x axis, and their related forces are an edge shear and bending moment.

It is possible to determine the edge shears and moments at the positions of the finite bridge boundaries given by the particular integral and superimpose upon this solution a complementary function such that the resultant at the bridge boundaries has the forces of a free edge, namely zero shear and zero bending moment.

This combined solution is the required one describing a loaded bridge structure of finite width.

The form of the Levy series shows that the deflexion profile for a longitudinal section is of constant form for all values of its position f and one may be obtained from another by direct multiplication by an arithmetical coefficient, i.e.,

$$\frac{w\,(x,y_2)}{w\,(x,y_1)} = K$$

where K is a constant for given values of y_1 and y_2 dependent upon the particular bridge structure and a definite loading system.

If the deflexion profile $w\,(x,y_1)$ is taken as that due to the oad being uniformly distributed across the whole bridge

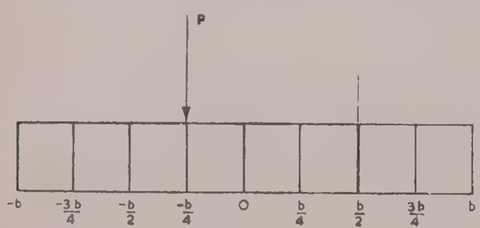


Fig. 10.

width then a set of distribution coefficient values K can be tabulated to provide the deflexion profiles for a number of transverse positions, i.e., values of $f = y_2$, for a number of loading systems.

It is evident that the values of these distribution coefficients will depend upon the transverse stiffness parameter θ and the torsion parameter α and a set will be required for many θ values combined with many α values.

Guyon has calculated K for a number of different section positions, f, and load positions, e, for a whole range of values for the case of zero torsion stiffness α.

Massonnet has continued the work by providing further K values with non-zero values of α.

3. The Application of the Methods of Distribution Coefficients

1 In 1946 Guyon evolved his method of distribution factors for simple grillage systems in which no torsional restraint exists. In 1950 Massonnet extended the method to embrace all systems irrespective of the torsional restraint.

Each method reduces the actual system to an equivalent system which is continuous in each direction such that the distribution in one direction does not contribute anything to the strength of the bridge in the orthogonal direction.

Massonnet gives a combined theoretical error of 3 per cent. arising from this reduction to an equivalent system (Fig. 2).

It is seen that the effective width of the equivalent continuous bridge must exceed the actual width by an amount equal to the spacing of the main beams. Since all the calculated values are based on the width of the effective system the transverse positions of the actual main beams must also be reduced to their effective positions. Thus,

$$\text{Effective beam (or load) position} = \left(\frac{\eta - 1}{\eta}\right) \times \text{actual}$$

beam position, where η is the number of main beams.

Thus, in the five beam grillage of Fig. 2 the effective beam positions are $+\,0.8b$, $+\,0.4b$, 0, $-\,0.4b$ and $-\,0.8b$ respectively.

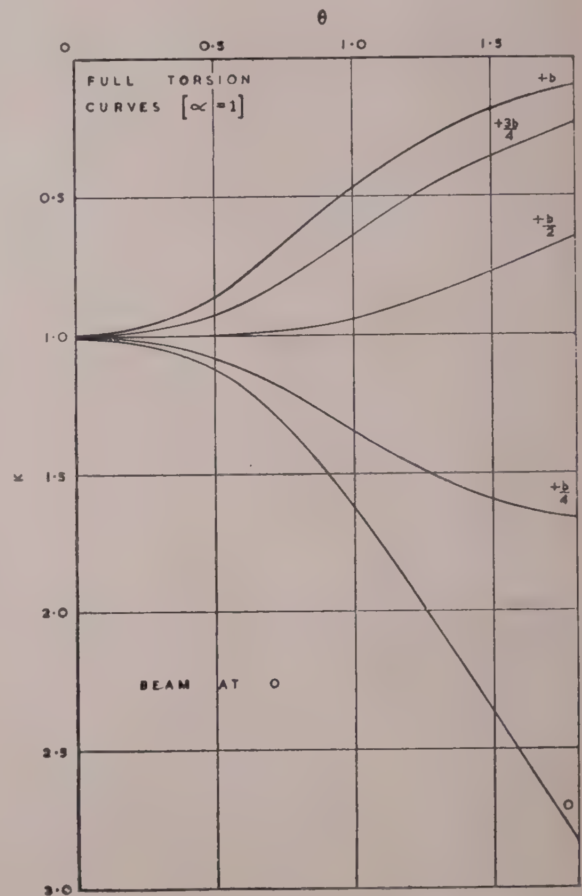


Fig. 11.—Full torsion distribution coefficients for a beam at $f = 0$ (due to Massonnet)

On the other hand, the equivalent width of a slab is equal to the actual width.

The equivalent width of a Tee-beam bridge is also identical with the actual width if this is measured as the distance between the lateral extremities of the slab.

A rigidity factor θ, which applies whether or not torsional restraint is present, defines the relative flexural rigidities of the bridge structure in the two principal directions. It is given by the relationship

$$\theta = \frac{b}{2a}\sqrt[4]{\frac{Iq}{Jp}}$$

where

$2b$ = the effective width

$2a$ = the effective span

In a grillage without torsion the value of θ alone determines the distribution of an applied load throughout the system.

In a grillage with torsion a further parameter α must be introduced to define the torsional rigidities per unit

length and width relative to the flexural rigidities in these directions. The parameter is given by

$$\alpha = \frac{G\left(\dfrac{I_0}{p} + \dfrac{J_0}{q}\right)}{2E\sqrt{\dfrac{IJ}{pq}}}$$

The torsional rigidities of most forms of section such as T's, I's, boxes etc., may be obtained approximately from simple formula given in standard text books.[8]

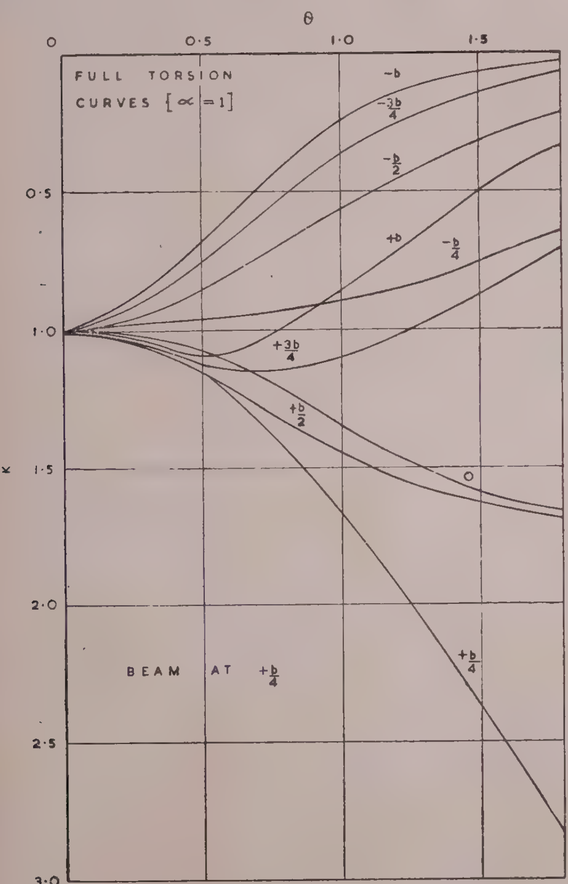


Fig. 12.—Full torsion distribution coefficients for a beam at $f = \frac{b}{4}$ (due to Massonnett)

The distribution coefficients K were developed as the relationships between the actual deflexion of a longitudinal section of a bridge under some loading system and the deflexion of the bridge with the loading system distributed uniformly across the bridge width, Fig. 3, i.e.

$$w = Kw_{av}$$

when w_{av} is the mean deflexion.

In the same way the moments in the longitudinal beams may be related to the mean moment through the distribution coefficients. Thus

$$M = KM_{av}$$

where M_{av} is the mean moment.

The curves of Figs. 4 to 9 give the values for K for a wide and continuous range of values of θ for systems without torsional resistance, i.e., $\alpha = 0$. Each group of curves gives the value of K for a given longitudinal section position f under loads whose eccentricity e varies from $+b$ to $-b$. It is important to note that both f and e are expressed in terms of the effective width $2b$.

For example, referring to Fig. 10 if $\theta = 1$ the distribution coefficients for a beam at $+\frac{b}{2}$ due to a load whose eccentricity varies from $-b$ to $+b$ are obtained from the curves of Fig. 6 and Table 1.

In an identical way the values of K at the other "beam" positions are taken from the appropriate groups of curves (Figs. 4 to 9).

The combined results can then be represented in tabular form as shown in Table 2. By Maxwell's theorem of reciprocity the effect of a load of eccentricity $e = \varphi$ on a beam of eccentricity $f = \beta$ is identical with the

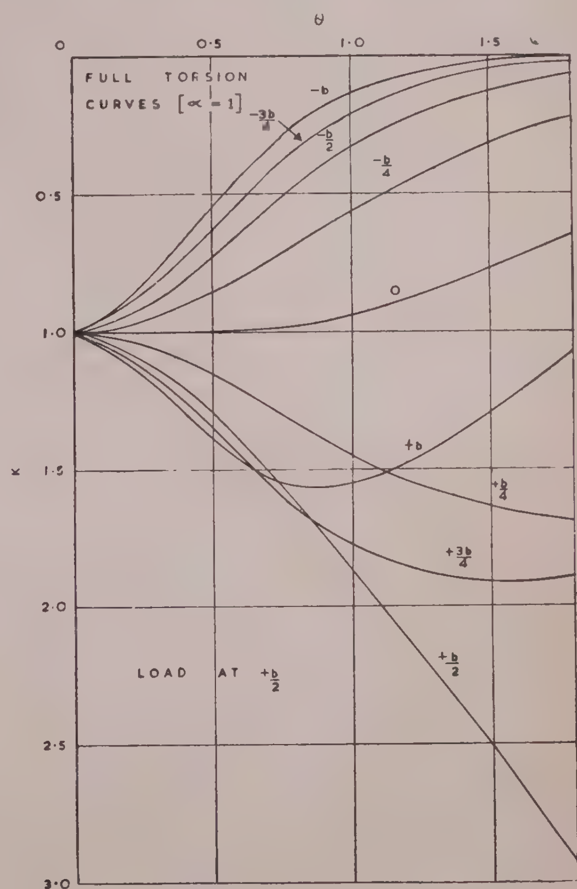


Fig. 13.—Full torsion distribution coefficients for a beam at $f = \frac{b}{2}$ (due to Massonnett)

effect of an equal load of eccentricity $e = \beta$ on a beam of eccentricity $f = \varphi$.

Thus, the Table of K values must be symmetrical about the marked diagonals.

If the load P is distributed uniformly over the transverse section all points on the transverse section will deflect equally and, hence, the values of the distribution coefficient will be everywhere equal to unity.

It follows that for all eccentricities of the load the average value of K must be unity. Using the mid-ordinate rule for the nine points considered

$$\left(-b, -\frac{3b}{4}, \quad . \quad . \quad . \quad . \quad . +\frac{3b}{4}, +b\right)$$

$$\sum_{n=2}^{n=8} K(n) + \frac{K_{(1)} + K_{(9)}}{2} = 8$$

TABLE 1.—Unit load distribution coefficients for beam at

$$f = +\frac{b}{2}$$

Load Position	$-b$	$-\frac{3b}{4}$	$-\frac{b}{2}$	$-\frac{b}{4}$	0	$\frac{b}{4}$	$\frac{b}{2}$	$\frac{3b}{4}$	b
K	−0.34	−0.16	0.03	0.40	1.0	1.85	2.36	2.00	1.26

TABLE 2.—Unit load distribution coefficients for all beam positions

Position of beam	Position of Load								
	$-b$	$-\frac{3b}{4}$	$-\frac{b}{2}$	$-\frac{b}{4}$	0	$\frac{b}{4}$	$\frac{b}{2}$	$\frac{3b}{4}$	b
0	−0.70	+0.17	+1.00	+1.90	**+2.30**	+1.90	+1.00	+0.17	−0.70
$\frac{b}{4}$	0.59	-0.12	0.40	**+1.08**	+1.90	**+2.36**	+1.85	+0.85	−0.28
$\frac{b}{2}$	0.35	0.16	**0.03**	+0.40	+1.00	+1.85	**+2.36**	+2.00	+1.26
$\frac{3b}{4}$	−0.09	**−0.16**	−0.16	−0.12	+0.17	+0.85	+2.00	**+3.35**	+4.30
b	**+0.15**	−0.09	−0.35	−0.59	−0.70	−0.28	+1.26	+4.30	**+8.80**

TABLE 3.—Unit load distribution coefficients for combined unit loads derived from single load coefficients

Beam Position	Single load coefficients unit load at			Combined load coefficients unit load at	
	$+b$	$+\frac{3b}{4}$	$+\frac{b}{2}$	$\frac{3b}{4}, \frac{b}{2}$	$\frac{3b}{4}, \frac{b}{2}, b$
b	0.16	−0.07	−0.34	−0.20	−0.02
$-\frac{b}{2}$	−0.34	−0.16	0.03	−0.06	−0.23
$+\frac{b}{2}$	1.26	1.98	2.36	2.17	1.71
$+b$	8.80	4.50	1.26	2.88	5.83

TABLE 4.—Distribution coefficients for the reduced applied loading of Figure 16

Load Position	$\frac{b}{4}$	$\frac{b}{2}$	$\frac{3b}{4}$	b	Multiplying Factor
$-b/2$	0.61	0.21	−0.17	0.53	0.88
$-b/4$	0.95	0.61	0.28	0	0.64
0	1.20	1.00	0.78	0.52	1.36
$b/4$	1.40	1.43	1.35	1.39	0.64
$b/2$	1.40	1.8	2.09	2.31	1.36
$3b/4$	1.43	2.09	2.85	3.6	0.64
b	1.39	2.30	3.6	4.8	0.48
ΣP K	7.13	7.73	8.35	8.88	

where $K(n)$ is the value of the distribution coefficient for the nth beam measured from an edge beam.

These two checks are of great assistance in detecting faults in reading values from the curves.

When the bridge does possess torsional strength, i.e., $\alpha \neq 0$ the Table of K is derived as follows.

Initially, the values of K_o, i.e., K for $\alpha = 0$, are tabulated in exactly the same way as in the example discussed above.

A Table of values of K_1, i.e., K for $\alpha = 1$, is then constructed being the coefficients relative to a system with full torsion, that is, for an anisotropic slab. The curves, which are identical in use with the curves for $\alpha = 0$, are

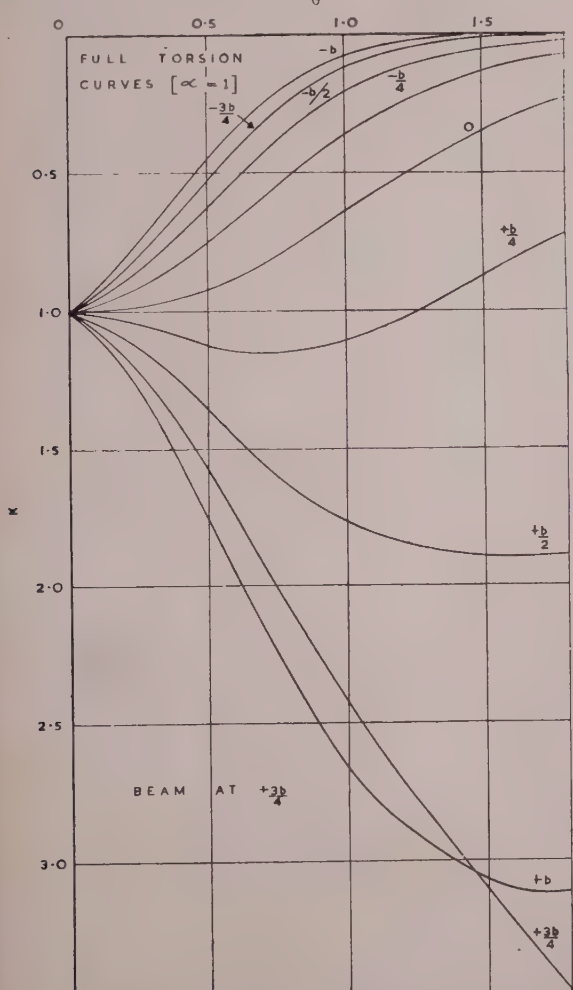


Fig. 14.—Full torsion distribution coefficients for a beam at $f = \frac{3b}{4}$ (due to Massonnet)

given in Figs. 11 to 15 and are derived from values given by Massonnet.

It has been found that the required values of $K\alpha$ for an actual beam and slab bridge in which α has a value other than 0 or 1 can be determined with sufficient accuracy by the interpolation

$$K\alpha = K_o + (K_1 - K_o)\sqrt{\alpha}$$

This interpolation is extremely valuable and ensures that the Table of $K\alpha$ is determined with equal ease for all values of α.

In this way the Table of coefficients $K\alpha$ is obtained for a " torsion " bridge.

The actual values of deflexion and bending moment are obtained by multiplying these K values by the " mean " deflexions or moments which are constant across any given transverse section. Hence, the coefficients for any load are proportional to the elastic transverse deformation profile of the equivalent system.

As yet only single loads have been considered. Under a number of loads $P_1, P_2, \ldots P_n$ acting on a given transverse section the resultant K factors are found by superimposing the factors for each single load.

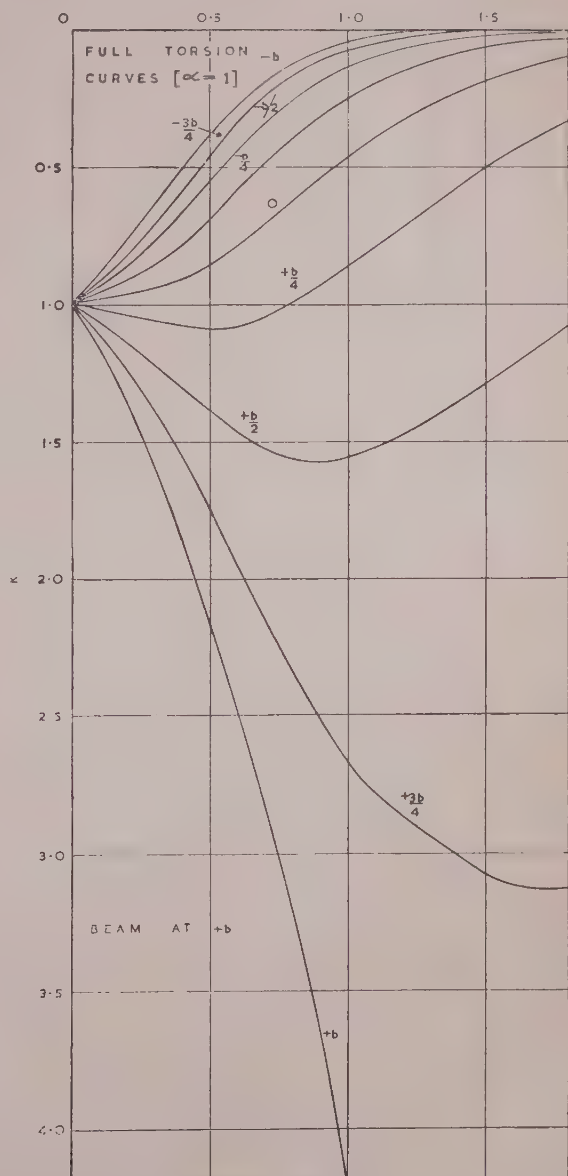


Fig. 15.—Full torsion distribution coefficients for a beam at $f = b$ (due to Massonnet)

As an example, the distribution coefficients for equal loads at a number of points are obtained by averaging the coefficients. Table 3 is obtained from Table 2 in this manner.

The " mean " effects are found by the uniform distribution of each individual load at the transverse section at which it is applied. The calculations then involved are in the determination of the deflexion or moment profile of a beam or slab section carrying a longitudinal distribution of point loads.

In a given bridge it is obviously unlikely that the equivalent beam positions will all correspond to the standard positions $-b, -\frac{3b}{4}, +b$, for which the curves have been drawn.

The same is true for the equivalent positions of the loads.

The required coefficients at the beam positions are then determined by drawing the transverse distribution influence curves from the Table of coefficients determined for the standard positions and, using the theorem of reciprocity, drawing in the curves for the required equivalent load and beam positions. The unit coefficient values are taken from these interpolated curves and the final coefficients are obtained by superposition as above.

Alternatively, the loads may be initially distributed to the standard positions $-b, \ldots + b$ and the required coefficients taken from the curves for the standard load positions by the vertical intercepts at the equivalent beam positions. This obviates the necessity of interpolating additional influence lines.

As an example, a transverse distribution of equal loads is, say, resolved into the components of Fig. 16 acting at the standard equivalent longitudinal sections.

As the loads are no longer equal each set of coefficients must be multiplied by the load to which it is due.

TABLE 5.—Values of the coefficient $10^4\mu_0$ ($\alpha = 0$, θ varying)

θ	Position of Section	Load Eccentricity								
		$-b$	$-3b/4$	$-b/2$	$-b/4$	0	$b/4$	$b/2$	$3b/4$	b
0	0	−2500	−1250	0	+1250	+2500	+1250	0	−1250	−2500
	$b/4$	−1758	−968	−178	+612	+1403	+2195	+487	−1221	−2930
	$b/2$	−937	−546	−156	+234	+625	+1055	+1406	−703	−2812
	$3b/4$	−273	−166	−58	+49	+156	+263	+371	+478	−1914
	b	0	0	0	0	0	0	0	0	0
0.669	0	−1418	−786	−103	+734	+1828	+734	−103	−786	−1418
	$b/4$	−873	−564	−234	+230	+826	+1685	+363	−817	−1920
	$b/2$	−435	−299	−175	+470	+266	+649	+1306	−454	−2123
	$3b/4$	−109	−86	−60	+25	+40	+106	+331	+559	−1681
	b	0	0	0	0	0	0	0	0	0
1.495	0	+16	−82	−161	−31	+758	−31	−161	−82	+16
	$b/4$	+45	−21	−88	−155	−26	+764	−26	−178	−150
	$b/2$	+12	+10	−33	−81	−140	+3	+795	−106	−562
	$3b/4$	−6	+60	−4	−24	−49	−75	+81	+583	−975
	b	0	0	0	0	0	0	0	0	0
2.34	0	+13	−5	−43	−99	+490	−99	−43	−5	+13
	$b/4$	−11	+10	−5	−43	−98	+490	−100	−44	+17
	$b/2$	−75	−9	+14	−5	−43	−99	+488	−98	−0
	$3b/4$	+62	−10	−12	+9	−7	−38	−77	+491	−431
	b	0	0	0	0	0	0	0	0	0
3.162	0	0	+2	−8	−71	+364	−71	−8	+2	0
	$b/4$	−117	+6	+6	−9	−71	+372	−75	−13	+127
	$b/2$	+196	−8	−4	+4	−8	−72	+366	−66	−184
	$3b/4$	−122	+2	+3	+1	+2	−8	−70	+370	−117
	b	0	0	0	0	0	0	0	0	0

TABLE 6.—Values of the coefficient $10^4 . \mu_1$ for $\alpha = 1$ and θ varying

θ	Position of Section	Load Eccentricity								
		$-b$	$-3b/4$	$-b/2$	$-b/4$	0	$b/4$	$b/2$	$3b/4$	b
0	0	−2500	−1250	0	+1250	+2500	+1250	0	−1250	−2500
	$b/4$	−2345	−1407	−469	+469	+1407	+2345	+938	−938	−2345
	$b/2$	−1875	−1250	−625	0	+625	+1250	+1875	0	−1875
	$3b/4$	−1093	−781	−469	−157	+157	+469	+781	+1093	−1093
	b	0	0	0	0	0	0	0	0	0
0.668	0	−419	−322	−151	+213	+1096	+231	−151	−322	−419
	$b/4$	−311	−240	−243	−108	+246	+1081	+179	−261	−528
	$b/2$	−213	−221	−225	−199	−86	+239	+1025	+30	−591
	$3b/4$	−117	−130	−145	−156	−141	−54	+204	+837	−498
	b	0	0	0	0	0	0	0	0	0
1.057	0	−121	−131	−117	+41	+740	+41	−117	−131	−121
	$b/4$	−66	−84	−107	−106	+43	+827	+20	−160	−207
	$b/2$	−35	−49	−72	−101	−106	+33	+705	−47	−317
	$3b/4$	−16	−24	−38	−62	−91	−99	+22	+623	−363
	b	0	0	0	0	0	0	0	0	0
1.495	0	−30	−47	−72	−30	+531	−30	−72	−47	−30
	$b/4$	−12	−21	−41	−38	−14	+529	−37	−87	−74
	$b/2$	−4	−8	−19	−40	−71	−34	+517	−67	−159
	$3b/4$	−2	3	−7	−18	−40	−72	−37	+475	−257
	b	0	0	0	0	0	0	0	0	0
1.778	0	−12	−24	−50	−44	+447	−44	−50	−24	−12
	$b/4$	−4	−9	−22	−49	−44	+446	−46	−57	−38
	$b/2$	−1	−3	−8	−22	−50	−46	+440	−64	−102
	$3b/4$	0	−1	−2	−8	−22	−51	−52	+410	−206
	b	0	0	0	0	0	0	0	0	0

This procedure then allows the true assessment of the effects of each load when superimposing unit results to find the coefficients which apply for a series of loads. This is illustrated in Table 4 which applies for the loading of Fig. 16.

Actually, it is often found that the coefficients for the standard positions ($-b$ . . . $+b$) are sufficient to

Fig. 16.—The reduction of a general arrangement of equal applied loads to the standard positions

indicate the values at the required positions by simple inspection the required values being obtained by linear interpolation.

4. The Bending Moments in the Cross Beams

The bending moments induced in the cross-members are a function of the relative deflexions of the main beams and, thus, are a function of the eccentricity of the load. The bending moment in the transverse direction is given by

$$M_y = -\frac{J}{q}\frac{d^2w}{dy^2}$$

This reduces to

$$M_y = \mu r b \sin\frac{\pi x}{2a}$$

where μ is a coefficient which is dependent on the values of θ, α, f and e and is analogous with the coefficient K in the longitudinal direction.

The values of $10^4\mu$ are given in Tables 5 and 6 for various values of θ for $\alpha = 0$ and $\alpha = 1$ respectively.

The interpolation formula

$$\mu_a = \mu_0 + (\mu_1 - \mu_0)\sqrt{\mu}$$

can again be used with sufficient accuracy for the intermediate values of α.

The partial load term r is defined, for a concentrated load, as

$$r = \frac{P}{a}$$

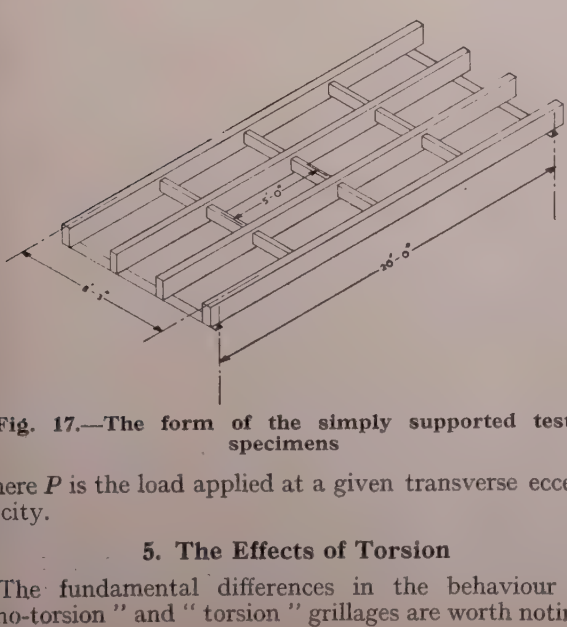


Fig. 17.—The form of the simply supported test specimens

where P is the load applied at a given transverse eccentricity.

5. The Effects of Torsion

The fundamental differences in the behaviour of "no-torsion" and "torsion" grillages are worth noting. A total lack of torsional restraint implies that the cross-beams are virtually attached to the main beams by pinned joints and that free rotation of the cross-beams can occur without inducing twisting in the main beams. Thus, as the flexural stiffness of the cross-connexion increases the movement of a transverse section in the vertical plane tends progressively to a linear-rotation

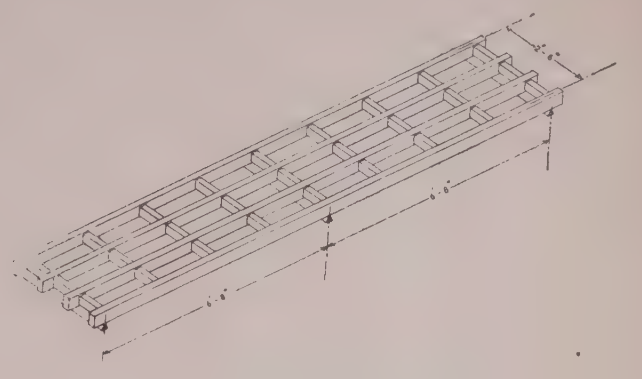


Fig. 18.—The form of the continuous test specimen

about the longitudinal axis. In the limit when the cross-connexion is infinitely rigid, i.e., $\theta = 0$ an eccentric load will cause a perfectly linear transverse rotation whose amount depends on this eccentricity. An analogy can be drawn between this limiting case of $\theta = 0$ and the stress

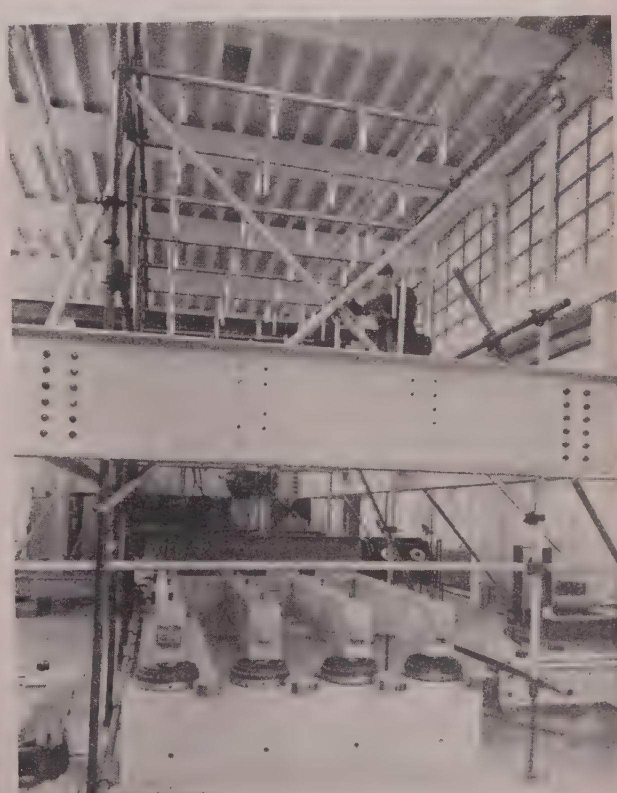


Fig. 19.—A general view of the testing arrangement of the simply supported specimens

diagram for a prestressed beam. At the neutral axis of a rectangular beam the stress remains constant at $\frac{P}{A}$ irrespective of the position of the centre of pressure; at the longitudinal axis of a bridge for which $\theta = 0$ the

value of K at the axis (i.e., beam at $f = 0$) is invariably equal to unity irrespective of the eccentricity of the applied load.

As long as the centre of pressure in the prestressed beam lies within the middle-third no tensile stresses will occur in the beam; as long as the eccentricity of the load resultant on the bridge is within the middle third of the effective width no hogging moments will be induced in any beam.

As the centre of pressure moves from the middle-third to the extreme of the section the maximum tensile stress will increase from 0 to -2 the maximum compressive stress increasing from $+2$ to $+4$. Similarly as the eccentricity of the resultant load increases from $+\frac{b}{3}$ to $+b$ the coefficient at $-b$ will increase from 0 to -2 whilst that at $+b$ will increase from $+2$ to $+4$. Thus the analogy is complete. This effect is seen in Fig. 39 where distribution curves are shown for an infinitely rigid cross-connexion. As the flexural rigidity of the cross-connexion is decreased to a finite value, i.e., θ is increased, the hogging moments decrease and the positive moments increase but at a smaller rate. This effect is shown in Figs. 39, 37, 36 and 38. There is an optimum value of θ for the most efficient behaviour of the deck. This optimum value is determined by the transverse disposition of the loads but should always be found for that position which results in the greatest possible eccentricity of the load resultant as this represents the critical loading on a bridge. The optimum value of θ is such that the slope of the deflected transverse profile is a minimum between the two beams carrying the greatest loads the equivalent deflected profile being proportional to the superimposed value of K for single loads.

On the other hand the efficiency of the load distribution in a bridge with torsional restraint increases as θ decreases until at $\theta = 0$, there is a perfect distribution and K is equal to unity at all points of the bridge. This is reasonable when it is remembered that the torsional rigidity is a function of the flexural rigidity and increases or decreases with it. The fixity at the points of intersection ensures that the relative deflexions of the main beams are a function of the flexural rigidity and, hence, the torsional rigidity of the cross-connexion. When this torsional rigidity becomes infinite no relative rotation of the ends of the cross-beam is possible and, thus, the relative deflexions of the main beams become equal to zero and a common value of K equal to unity exists for each beam.

Thus, the optimum value of θ for a slab is invariably zero whilst the optimum value of θ for a "no-torsion" grillage depends on the form of loading and is zero only for symmetrical loading.

It follows that, for any bridge intermediate to these extremes, the optimum cross-connexion will lie between

TABLE 7.—Data relating to the test specimens

Deck Number	1	2	3	4
MAIN BEAMS				
No.	4	4	4	4
Span (ft.) (in.)	20—0	20—0	20—0	6—8
Depth (in.)	12	12	12	4
Width (in.)	6½	5	5	2
(Post-tensioning) Stressing method	Magnel-Blaton	Freyssinet	Freyssinet	Freyssinet
No. of 0.2 in. wires	16	12	12	2
Cable line	Parabolic	Parabolic	Parabolic	Straight
Maximum compressive stress (lb. in. $^{-2}$) after 15 per cent. loss	1,750	1,750	1,750	1,000
Zero tension bending moment lb. ft.	22,750	17,500	17,500	434
Average cube strength at stressing (lb. in. $^{-2}$)	6,250	5,750	5,750	5,850
Main Beam spacing (in.)	21½	33	33	10½
CROSS BEAMS				
No.	3	3	3	9
Length (ft. in.)	5—3¾	8—3	8—3	2—7½
Depth (in.)	12	7	9	2½
Width (in.)	6½	3	3	2
Cast	in situ	precast	precast	precast
Stressing method (Post-tensioned)	Lee-McCall	Magnel-Blaton	Magnel-Blaton	Screw Jack
Stressing units	one lin. rod	2, 4, or 6 0.2 in. wires	4, 6, or 8 0.2 in. wires	4 0.1 in. wires
Average prestress (lb. in. $^{-2}$)	up to 875	330, 670 or 1,000	520, 780 or 1,040	1,000
Cross beam spacing (ft. in.)	5—0	5—0	5—0	1—8

TABLE 8. Unit load distribution coefficients for the second specimen, $\theta = 0.584$, $\alpha = 0$

Position of beam	Position of Load								
	$-b$	$-3b/4$	$-b/2$	$-b/4$	0	$b/4$	$b/2$	$3b/4$	b
0	[illegible]	[illegible]	1.01	1.32	1.5	1.32	1.01	0.68	0.38
$b/4$	0.15	0.24	0.61	1.01	1.32	1.5	1.46	1.32	1.12
$b/2$	—0.[illegible]	0.17	0.21	0.61	1.01	1.48	1.86	2.07	2.22
$3b/4$	0.8	0.40	0.18	0.23	0.70	1.29	2.08	2.9	3.8
b	—1.1	[illegible]	0.52	0.15	0.36	1.11	2.22	3.8	5.2

zero and the "no-torsion" optimum value. The optimum value is given with sufficient accuracy by

$$\theta_a = \theta_0\, opt\, (1 - \sqrt{\alpha})$$

θ_0 *opt* is found quickly by trial as demonstrated in the third example at the end of the paper.

6. The Experimental Investigation

The methods of analysis which have been discussed in some detail in the foregoing pages analyse the behaviour of an equivalent uniform system rather than of the actual system under consideration.

It was therefore felt that before the methods could be recommended for design, tests would have to be carried out on models and actual structures to ensure that the calculations do predict the actual behaviour of the structure. The experimental results reported here relate to tests on simple grillage structures of prestressed concrete extending over one or two spans.

It is realized that this is only the first step and structural models incorporating slabs should be tested as well as full size bridges if possible. At the time of writing a new series of tests of Tee beam and box beam structures is being started.

The leading data relating to the test specimens is given in Table 7 and their form is indicated in Figs. 17 and 18. The following report of the tests necessarily gives the minimum of detail, further information of the test apparatus and results being included elsewhere.[9,10]

7. Tests on a Simply Supported Specimen

The three single span grillages each consisted of four main beams interconnected by three sets of cross-beams

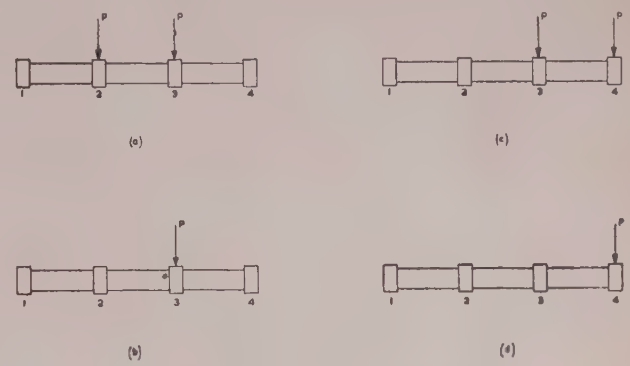


Fig. 20.—Sequence of loading

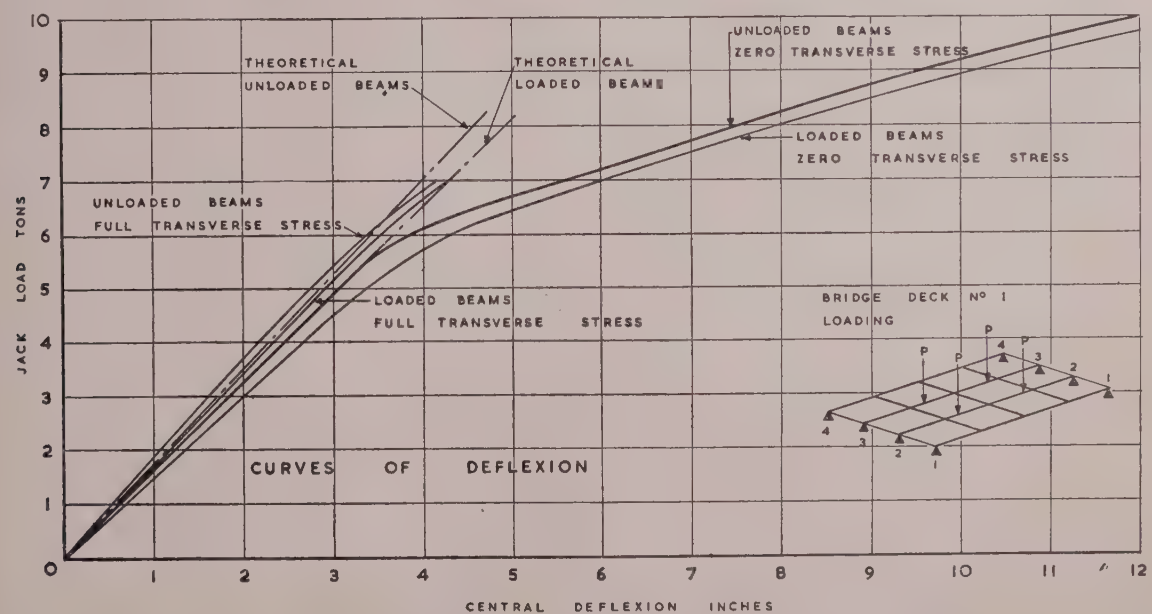


Fig. 21.—Comparison between the actual and the theoretical deflexions for the first test specimen

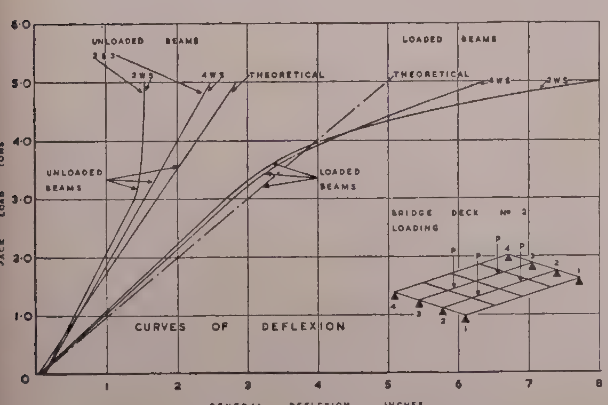


Fig. 22.—Comparison between the actual and the theoretical deflexions for the second specimen

at the $\frac{1}{4}$-points and at mid-span. The relative dimensions of the main beams and diaphragms were chosen so that the effective stiffness of a specimen, as measured by the stiffness parameter θ, differed for each grillage.

Provision was made to adjust the amount of prestress in the cross-beams as it was felt that the prestressing force would affect the effective transverse flexual stiffness. This was achieved in the first specimen by stressing transversely on the Lee-McCall system and in the other two grillages by stressing on the Magnel system. A complete series of tests was made with two, four and six 0.2 in. diameter wire cables stressed in the transverse direction for the second specimen and with four, six and eight wire cables similarly stressed for the third deck.

In the second and third decks the cross-beams were precast in segments and were held in position by mortared joints before the stressing was carried out. The cross-beams of the first deck were cast in situ. As the prestress was to be increased subsequently the transverse cables were not grouted.

The main beams were supported on fixed abutments at one end with a line contact. At the other end each beam rested on a pressure capsule of 50-ton capacity. It

was hoped that the readings on these capsules would indicate the distribution of load throughout the system at various eccentricities of load.

The load was applied by hydraulic jacks of 50-ton capacity at a transverse section distant 7 ft. from either end of the grillage and measured by 50-ton capsules identical with those used at the abutments (Fig. 19). The jack load was distributed to the required beams by an R.S.J. and bearing blocks.

Deflexions were measured at nine equally spaced points on each beam by scales and verniers which were attached to a frame of tubular scaffolding completely independent of the main test frame. An accuracy of 0.01 in. in deflexion readings was expected from this arrangement.

Strains were recorded over a gauge length of 8 in. by a mechanical gauge evolved at the laboratory. The expected accuracy of these readings was 1×10^{-5}. Measurements by electrical resistance strain gauges was attempted in the first specimen but after trouble had arisen due to drift in the gauges full dependence was placed on the mechanical strain gauge in subsequent tests.

The various load combinations used in the tests are shown in Fig. 20. The loading was always symmetrical about the transverse axis of the bridge and, hence, the same " mean " deflexion and bending moment profiles could be used for the complete series of tests on any one specimen. The second and third specimens varied only in the dimensions of the cross-connexion and, hence, the " mean " effects were common to both.

Each grillage was analyzed initially as a specimen without torsional strength in which case $\alpha = 0$. As each grillage consisted of four main beams of equal flexural stiffness, the effective width of each specimen was equal to 4/3 of the actual width and the effective beam positions were

$$-\frac{3b}{4}, -\frac{b}{4}, +\frac{b}{4} \text{ and } +\frac{3b}{4} \text{ respectively.}$$

The analysis of the second specimen is given later as Example 1 which also includes the distribution Tables for the other decks under the load sequence of Fig. 20. The values of θ of 0.248, 0.584 and 0.484 for the three specimens reflect the differences in the relative transverse rigidities.

8. Results

The first specimen served mainly to check the capacity and the suitability of the various pieces of test equipment. Consequently, only tests for symmetrical loading, i.e. either the two inner beams or the two outer beams loaded equally, were made. Yet the difference between the behaviour of the deck for zero transverse prestress

TABLE 9.—Combined distribution coefficients for the second specimen, $\theta = 0.584$, $\alpha = 0$

Position of beam	Load Position $+\frac{b}{4}$	$+\frac{3b}{4}$	$-\frac{b}{4}, +\frac{b}{4}$	$+\frac{b}{4}, +\frac{3b}{4}$
$-\frac{3b}{4}$	0.24	—0.49	0.78	—0.125
$-\frac{b}{4}$	1.01	0.21	1.25	0.61
$+\frac{b}{4}$	1.50	1.29	1.25	1.39
$+\frac{3b}{4}$	1.32	2.9	0.78	2.11

TABLE 11.—Distribution coefficients for the third specimen for the actual loading arrangements adopted in the tests, $\theta = 0.454$, $\alpha = 0$

Position of beam	Load Position $+\frac{b}{4}$	$+\frac{3b}{4}$	$-\frac{b}{4}, +\frac{b}{4}$	$+\frac{b}{4}, +\frac{3b}{4}$
$-\frac{3b}{4}$	0.32	—0.53	0.87	—0.105
$-\frac{b}{4}$	0.95	0.29	1.150	0.62
$+\frac{b}{4}$	1.36	1.38	1.150	1.37
$+\frac{3b}{4}$	1.42	2.82	0.87	2.12

TABLE 10.—Distribution coefficients for the first specimen for the actual loading arrangements adopted in the tests, $\theta = 0.248$, $\alpha = 0$

Beam Position	Load Position (1) $+\frac{b}{4}$	(2) $+\frac{3b}{4}$	(3) $-\frac{b}{4}, +\frac{b}{4}$	(4) $+\frac{b}{4}, +\frac{3b}{4}$
Edge Beam (1) $-\frac{3b}{4}$	0.4	—0.63	0.96	—0.115
Inner Beam (2) $-\frac{b}{4}$	0.86	0.37	1.035	0.615
Inner Beam (3) $+\frac{b}{4}$	1.21	1.48	1.035	1.345
Edge Beam (4) $+\frac{3b}{4}$	1.53	2.73	0.96	2.13

and the full prestress of 875 lb. per sq. in. are worth noting. The working load for the symmetrical load position was 5.6 tons. The test at full transverse prestress was carried out first. The agreement between the practical and the theoretical results in the linear elastic range was acceptable as shown in Fig. 21. The curve of continued to take their share of the load throughout simply because the joints of the cast in situ cross beams with the main beams did not fail.

On the other hand reference may be made to the results for the symmetrical loading of the second deck when only two wires were stressed transversely which

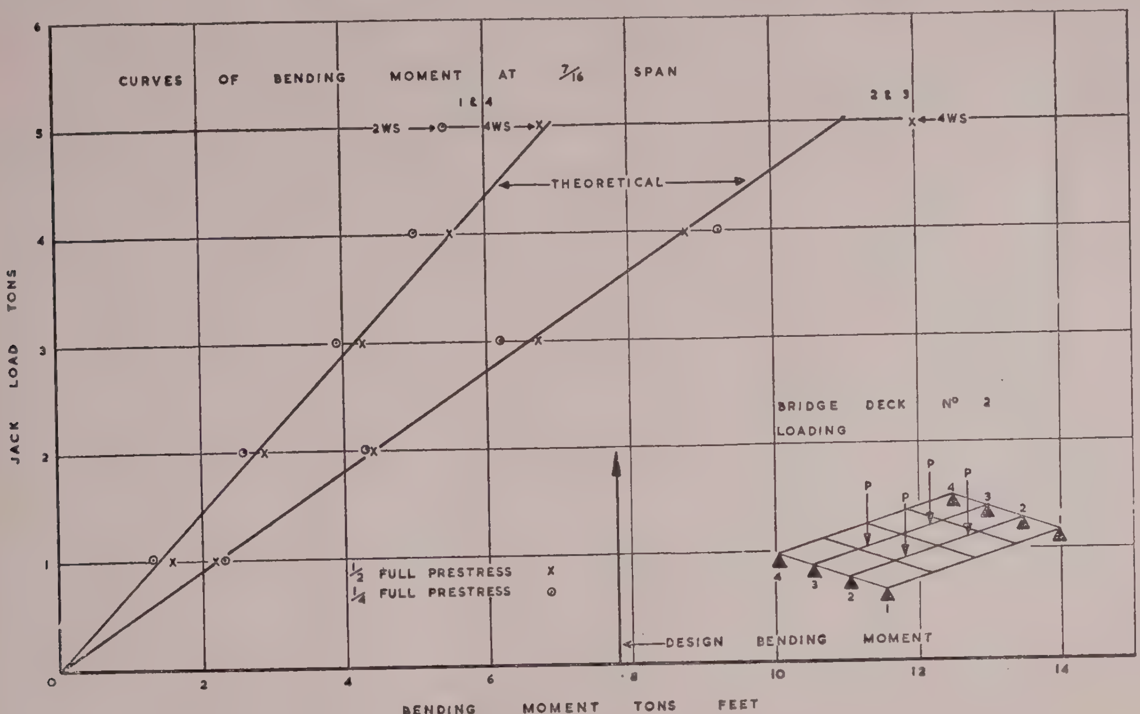


Fig. 23.—Comparison between the moments calculated from measured strain readings and from theory for the second specimen

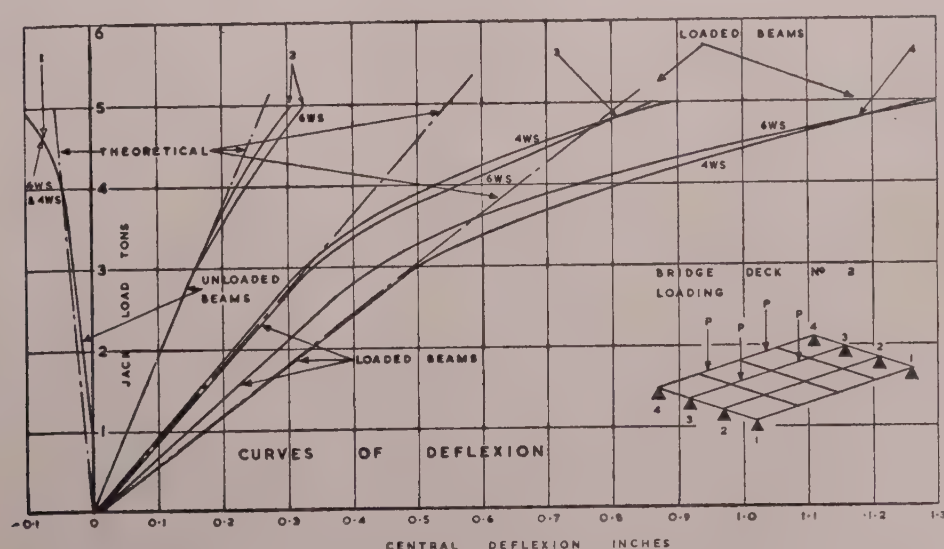


Fig. 24.—Comparison between the actual and the theoretical deflexions for the second specimen

deflexion became non-linear at a load just in excess of 6 tons as might be expected. The test at zero transverse prestress was carried out after the main beams had been cracked in previous loadings. Thus, the deflexion curve became non-linear at a smaller load than before and approximated to the working load. Thereafter the rate of increase of deflexion increased greatly not only in the loaded beams but also in the unloaded beams. This is important in illustrating that the unloaded beams was equivalent to a uniform prestress of only 350 lb. per sq. in. (Figs. 22 and 23).

The main beams cracked at a little under the working load of 3.5 tons. Immediately afterwards the cross-beams broke away at the joints since the prestress was insufficient to maintain continuity under the increased amount of differential deflexion.

It follows that the capacity of the cross-beam to distribute the applied load to the outer beams decreased[9]

and, in fact, no additional load was taken by these beams. Thus, the main beams had to resist practically the whole of the subsequent load increments.

When the transverse prestress was doubled by stressing four 0.2 in. diameter wires the outer beams continued to take their share of the load throughout the test which extended to 40 per cent. overload with a corresponding 135 per cent. overload. This excessive load was allowed since such a loading approximated most nearly to the practical case of a vehicle passing over the bridge.

Crushing of the outer mid-span cross-beam between the loaded beams was noted but failure did not occur. Nevertheless, permanent damage must have been caused as this cross-beam failed at a load of only three tons, less than the working load, under the symmetrical loading of Fig. 20a. The cube crushing strength for the cross-beam at failure was 7,500 lb. per sq. in.

TABLE 12.—Unit load distribution coefficients for the continuous specimen $\theta = 0.387$, $\alpha = 0$

Beam Position	Load Position $-\frac{3b}{4}$	$-\frac{b}{2}$	$-\frac{b}{4}$	0	$\frac{b}{4}$	$\frac{b}{2}$	$\frac{3b}{4}$
0	0.90	0.99	1.10	1.15	1.10	0.99	0.90
$\frac{b}{4}$	0.38	0.62	0.90	1.10	1.24	1.37	1.48
$\frac{b}{2}$	—0.16	0.24	0.62	0.99	1.37	1.77	2.10
$\frac{3b}{4}$	—0.60	—0.16	0.38	0.90	1.48	2.10	2.76

decrease in the final deflexions and bending moments on the loaded beam.

This, and subsequent results, show that the amount of transverse prestress must be sufficient to prevent tension occuring in the cross-beam at the intersections up to the required maximum load.

On the other hand, when the transverse prestress is great as with the stresses of 1,000 lb. per sq. in. and 1,040 lb. per sq. in. in the second and third decks respectively, the danger seems to be as great as with under stressing since high compressive stresses are induced along the diagonal of a cross-beam segment due to the differential deflexion of the adjacent main beams. The increased prestress increases the effective flexural ridity of an edge beam in the horizontal plane. Thus, the ends of the cross-beam undergo a further restriction to rotation and considerable shear stresses are induced along a diagonal plane of the cross-beams.

TABLE 13.—Combined distribution coefficients for the continuous specimen $\theta = 0.387$, $\alpha = 0$

Beam Position	Load Position $+\frac{b}{4}$	$+\frac{3b}{4}$	$-\frac{b}{4}, +\frac{b}{4}$	$+\frac{b}{4}, +\frac{3b}{4}$
$-\frac{3b}{4}$	0.38	—0.60	0.93	—0.11
$-\frac{b}{4}$	0.90	0.38	1.07	0.64
$+\frac{b}{4}$	1.24	1.48	1.07	1.36
$\frac{3b}{4}$	1.48	2.76	0.93	2.12

The second bridge deck failed precisely in this way. The maximum loading of Fig. 24 represented almost

Thus, it is possible to overstress transversely as well as understress in the same way as it is possible to overstress or understress a normal beam. There is a critical value of the transverse stress, determined by the working load for a particular load position, which prevents failure at the joints without risking failure of the cross-beam itself. This critical value can be determined by using the coefficients μ, of Table 5 and inserting the value of the working load in equation 2.

In the second and third decks the most efficient behaviour was achieved by stressing four and six wires respectively. These correspond to prestresses of 670 lb. per sq. in. and 750 lb. per sq. in.

Overstressing has one other serious disadvantage. An edge beam of a bridge with little or no torsional strength will have to sustain hogging effects when the applied load is at a position of high eccentricity on the other side of the bridge. This is shown in Figs. 24, 25 and 26. These hogging moments increase for a given load as the value of θ diminishes and corresponds to a "maximum" negative value of K of -2 when $\theta = 0$. Thus, the smaller the values of θ the greater is the possibility that an edge beam lifts completely off its end supports, which, obviously, is highly undersirable. An increase in the transverse prestress above the critical amount was seen to increase the effective transverse flexural rigidity of the bridge. Thus θ diminished and the hogging effects were more pronounced. As the prestresses were then additive to the applied stresses a dangerous situation was incurred. The tests revealed permanent cracking at the upper surface of the edge beam, showing that the resultant stresses due to prestress and dead load were themselves tensile.

The bending moments were derived from the strain values by a control test on a single main beam of the third grillage which remained undamaged after the test to failure. The load was applied at 7 ft. from either end of the beam and the strains were recorded at five sections of the span.

The strain distribution in the compressive zones remained linear up to failure at an applied bending moment of 21 tons ft. compared with the design moment of 7.8 tons ft.

Calibration curves were drawn between bending moment and rotation which was measured as the ratio of the strain at the upper surface divided by the depth to the measured position of the neutral axis.

These calibration curves were applied to the actual results for the actual grillage tests and the moments deduced without the necessity for assuming a value of Young's Modulus.

9. Tests on a 2-Span Continuous Grillage

The relative success of these initial investigations influenced the decision to extend the tests to continuous specimens.

Accordingly, a 4-beam grillage was constructed to be continuous over two equal spans of 6 ft. 8 in. The 4 in. × 2 in. rectangular section beams were prestressed to a uniform stress of 1,000 lb. per sq. in. by two No. 0.2 in. diameter wires positioned along the axis of beam and stressed on the Freyssinet system. The cross-members of $2\frac{3}{4}$ in. × 2 in. rectangular section were precast in 8 in. sections. Interconnexion was made over the three supports and at the $\frac{1}{4}$, $\frac{1}{2}$ and $\frac{3}{4}$ point sections of each span. The overall width of only 2 ft. 6 in. made it virtually impossible to utilize any of the accepted stressing systems which must have resulted in high percentage losses. A screw jack arrangement was evolved to surmount the difficulties. Dynamometers each carrying two electrical resistance strain gauges in series measured the strain in the 0.1 in. diameter wires and, by calibration the stress. A uniform transverse prestress of 800 lb. per

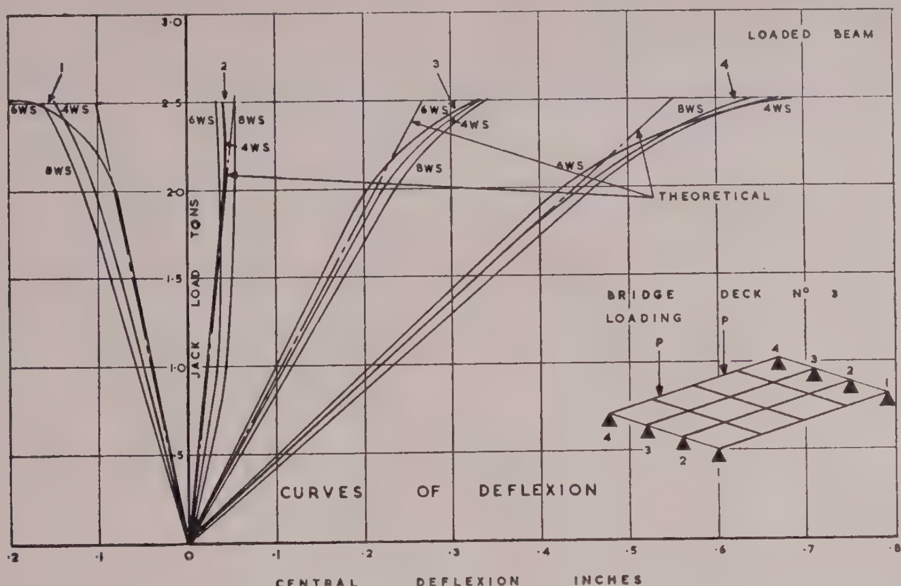


Fig. 25.—Comparison between the actual and the theoretical deflexions for the third specimen

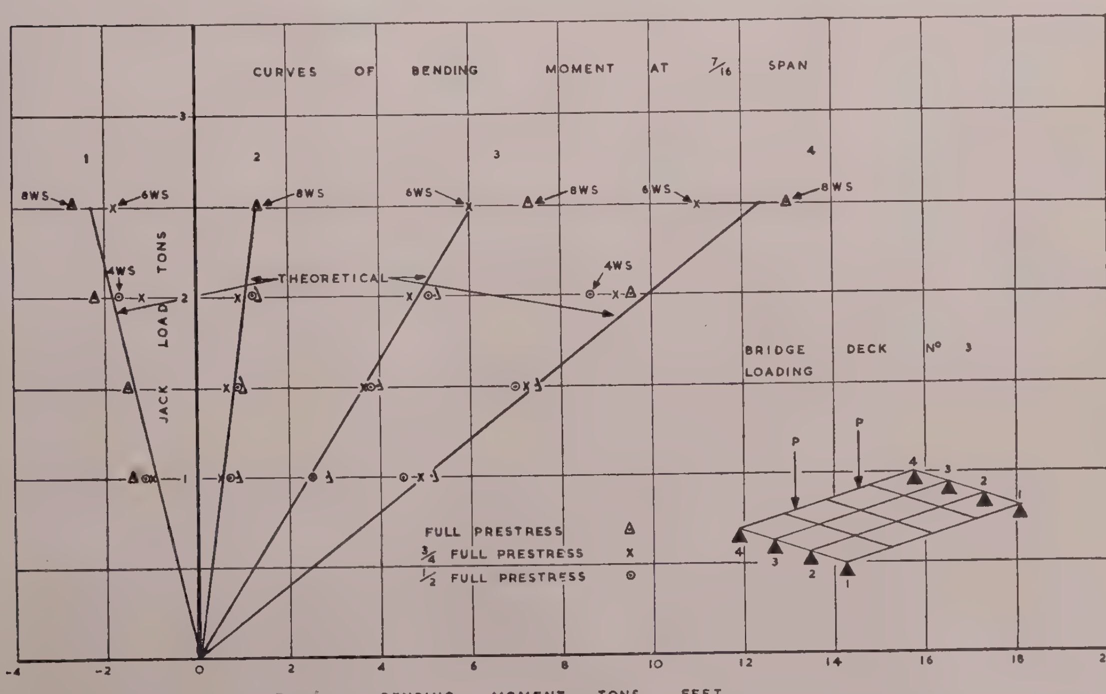


Fig. 26.—Comparison between the moments calculated from measured strain readings and from theory for the third specimen

sq. in. was achieved in this way. Unfortunately, drift of the dynamometers did not allow an accurate assessment of subsequent stress losses to be made. Therefore, a total loss of 15 per cent. was assumed.

The working loads for this specimen were low and to avoid errors a high sensitivity had to be achieved in the application of the loads. Fig. 27 illustrates the simple lever system which was evolved to give an applied load to the deck accurate to 5 lb. The load was applied by hydraulic jacks of 10-ton capacity and was gauged by pressure capsules. The initial lever ratio was 4.87 : 1. Subsequently when higher loads were required this lever ratio was reduced first to 3.41 : 1 and finally to 2.086 : 1 the reduction being dictated by the strength properties of the lever arm itself. The load was transmitted to the beam, at mid-span by the loading beams of Fig. 28. The knife edge bearings ensured the application of equal loads to each beam and allowed the free transverse rotation necessary because of the differential deflexions of the beams.

The load distribution throughout the system was recorded by dial gauges accurate to 0.001 in. which were fixed to a rigid frame placed under the specimen and independent of it.

The theoretical distributed moments and deflexions are functions of two parameters, viz. the "mean" effects and distribution coefficients K. The fundamental aim of the investigations was to measure K and, hence, it was necessary to measure the "mean" deflexions directly. By multiplying the measured "mean" effects by the theoretical coefficients and comparing these with the experimental results, discrepancies could be said to be due almost wholly to the assumptions of analysis. The direct measurement is especially important when basing the investigations on deflexion values which are influenced so greatly by the value of Young's modulus in the actual structure and which is not necessarily constant for all loading stages. The deflected profiles obtained when all four beams were loaded equally at the end span section of span X are given in Fig. 29.

On the first analysis it was found that the effective value of E varied at different points in the span. However, measurements at the central support showed that some settlement had occurred during the test. Fortunately this settlement was found to be proportional to the load.

A continuous beam calculation including the effects of the settlement showed that the mean deflexions were of the correct form along the span and the value of E was found to be 4.25×10^6 lb. in.[2]

Settlement occurred in all tests and, hence, the measured mean values were used without correction.

The tests for single span loading showed most clearly the validity of the Massonnet coefficients for this specimen indicating the existence of torsional restraint at

TABLE 14.—Unit load distribution coefficients for the continuous specimen $\theta = 0.387$, $\alpha = 1$

Beam Position	Load Position						
	$-\frac{3b}{4}$	$-\frac{b}{2}$	$-\frac{b}{4}$	0	$\frac{b}{4}$	$\frac{b}{2}$	$\frac{3b}{4}$
0	0.920	0.998	1.070	1.120	1.070	0.998	0.920
$\frac{b}{4}$	0.797	0.873	0.965	1.070	1.140	1.120	1.103
$\frac{b}{2}$	0.705	0.772	0.873	0.998	1.120	1.213	1.260
$\frac{3b}{4}$	0.710	0.705	0.797	0.920	1.103	1.260	1.374

TABLE 15.—Unit load distribution coefficients for the continuous specimen $\theta = 0.387$, $\alpha = 0.438$

Beam Position	Load Position						
	$-\frac{3b}{4}$	$-\frac{b}{2}$	$-\frac{b}{4}$	0	$\frac{b}{4}$	$\frac{b}{2}$	$\frac{3b}{4}$
0	0.913	0.996	1.080	1.130	1.080	0.996	0.913
$\frac{b}{4}$	0.657	0.788	0.943	1.080	1.174	1.205	1.230
$\frac{b}{2}$	0.413	0.593	0.788	0.995	1.205	1.400	1.540
$\frac{3b}{4}$	0.270	0.413	0.657	0.910	1.230	1.540	1.840

the intersections. The close agreement is shown in Figs. 30 and 31 for the loaded span.

It is important to note that the theory is not valid in the unloaded span. The distribution properties break down at the internal support and the deflexion of all beams in the unloaded span are virtually equal no matter what the eccentricity of the applied load might be. These uniform deflexions are invariably equal to the

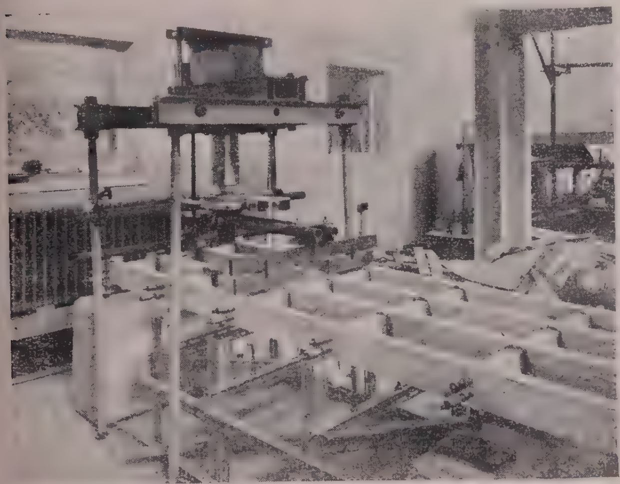


Fig. 27.—A view of one span of the continuous grillage showing lever loading system

initially measured "mean" deflexions of the unloaded span.

The explanation of this phenomenon appears to be as follows. In the loaded span differential deflexion is possible; at the support it is obviously not. The differential deflexions of the loaded span must cause unequal mid-support bending moments. The cross beams at the central support, redistribute these unequal moments to each beam apparently through their torsional stiffness. The closeness to which the redistribution approaches the limit of equal loads in each beam, is a function of the torsional stiffness of the cross-beams at the support. In this specimen the deflexions of each beam of the unloaded span were so nearly equal as to be considered equal, see Figs. 32 and 33. The secondary effects of the redistribution do not apparently extend far into the loaded span as shown by the excellent agreement between the measured deflexions at $\frac{3}{4}$ span and the deflexions based on the Massonnet coefficients.

Two-span loading gives the results of Figs. 34 and 35 and are forecast with agreeable accuracy by superimposing the results of single span loading.

At no time during the investigations did the applied load greatly exceed the working load. This was because the specimen was required for a large range of load eccentricities in an undamaged state.

The load factor was gauged by the final test to destruction in which the load was applied directly to the bridge on beams 1 and 2 of one span and beams 3 and 4 of the other. This loading arrangement approximated most nearly to the case of two abnormal loads acting on an actual beam structure and which would result in an appreciable twisting moment in the deck.

From the observed "mean" deflexions of the specimen and the calculated distribution factors the working load was determined as 960 lb. at each jack.

The majority of the dial gauges had to be removed in case of an abrupt failure but the deflexions of the mid-span sections of the two outer beams were taken.

The first deviation from the linear load-deflexion relation was for beam 4 at a jack load of 1,400 lb. and visible bending cracks appeared in this beam, under the load, at 1,500 lb. with a deflexion of 0.074 in. compared with a theoretical deflexion of 0.060 in. The factor of safety of the bridge against cracking under the given disposition of the applied loads was thus 1.460.

The working load for the bridge, calculated such that the load distribution effects are ignored, was 1,620 lb. which gives the factor of safety against cracking as 0.865.

The crack in beam 4 developed until at 1,800 lb. the initial cracking of beam 1 occurred under the load.

At 2,000 lb. the bending crack in beam 4 had developed considerably and in addition a shear crack had formed up to the cable position. This load also corresponded to the first apparent bending crack at the mid-span of beam 3. No failure was apparent in any of the cross-beams or at any of the joints.

The cracking of span Y continued with further increase of load. No further cracks or development of existing cracks had appeared in span X.

At 2,350 lb. the mid-span deflexion of beam 4 was 0.114 in. as against a theoretical deflexion of 0.087 in. The first combined bending and torsion cracks were visible in all the beams of the span adjacent to the central support.

At 3,000 lb. serious crushing had occurred in the beams 3 and 4 at mid-span. Excellent specimens of pure torsion cracks were apparent near the end supports of beams 2 and 3 in span Y. These were due to the restriction to the free transverse rotation of the beams by the negative reaction at the end supports.

The structure failed in span Y with 3,350 lb. on span Y and 3,450 lb. on span X the required hinge system being

Fig. 28.—An exploded view of the subsidiary loading leavers showing the parts used for five different loading arrangements

caused by combined bending and torsion at the mid-support, severe crushing in bending in beam 4 at mid-span, severe crushing in bending and torsion at mid-span of beam 3 and pure torsion near the ends of beams 3, 2 and 1 in span Y.

No damage was indurred to either the cross-beams or the joints even at failure.

The span X remained only slightly damaged and this was loaded to failure alone. The span failed in bending

due to the lack of continuity at the mid-support. The impact of the failure cracked two of the cross-beams along the cable axis.

The load factor for the structure was 3.49 and the safety against cracking was 1.49 both of which seem quite adequate.

It was satisfactory to note that the structure failed simultaneously along the whole length of span *Y* and not at one isolated section.

10. Conclusions

The main conclusions from these tests were that the simply supported grillages behaved as "no-torsion" grillages and that the deflexions and the bending moments were forecast with agreeable accuracy by the method of distribution coefficients initiated by Guyon.

The continuous grillage on the other hand showed good agreement with torsion calculations based upon the analysis of Massonnet.

The required transverse prestress is that which will maintain the joints of the system up to the required working load of the grillage or for slight overload. The value of the working load will vary for different load eccentricities and the limiting condition must be determined for the maximum practical eccentricity.

If the transverse stress is too low premature failure will occur at the joints and the efficiency of the load distribution will be affected adversely.

If the transverse stress is too high the possibility of crushing of the cross-beams is increased. Also the effective transverse rigidity is increased and the hogging effect in an edge beam thus magnified.

Thus the criterion for the efficient behaviour of the system even after cracking of some of the main beams is that the joints must be maintained without the introduction of too high a compressive stress in the diaphragms.

The tests to failure revealed a load factor of 3.3 for the first specimen and 2.7 for the third specimen both of which failed by the fracture in bending of the most heavily loaded beam, and 3.49 for continuous specimen

TABLE 16.—Combined distribution coefficients for the continuous specimen $\theta = 0.387$, $\alpha = 0.438$

Beam Position	Load Position $+\frac{b}{4}$	$+\frac{3b}{4}$	$+\frac{b}{4}, -\frac{b}{4}$	$+\frac{b}{4}, +\frac{3b}{4}$
$-\frac{3b}{4}$	0.657	0.270	0.943	0.463
$-\frac{b}{4}$	0.943	0.657	1.058	0.800
$+\frac{b}{4}$	1.174	1.230	1.058	1.202
$+\frac{3b}{4}$	1.230	1.840	0.943	1.530

TABLE 17.—Combined distribution coefficients for the abnormal load of 180 tons at maximum eccentricity, θ varying, $\alpha = 0$

θ	$-b$	$-\frac{3b}{4}$	$-\frac{b}{2}$	$-\frac{b}{4}$	0	$+\frac{b}{4}$	$+\frac{b}{2}$	$+\frac{3b}{4}$	$+b$
0	—0.5	—0.12	0.25	0.62	1.00	1.37	1.74	2.21	2.50
0.2	—0.49	—0.13	0.25	0.62	0.99	1.36	1.73	2.13	2.49
0.4	—0.49	—0.13	0.23	0.61	0.99	1.36	1.75	2.12	2.45
0.6	—0.50	—0.14	0.21	0.61	1.07	1.43	1.80	2.10	2.25
0.8	—0.44	—0.15	0.18	0.55	1.04	1.53	1.91	2.08	2.14
1.0	—0.34	—0.14	0.09	0.45	1.02	1.67	2.07	2.07	1.79
1.2	—0.22	—0.13	—0.02	0.30	0.97	1.82	2.30	2.05	1.41
1.5	—0.01	—0.07	—0.10	0.08	0.83	2.02	2.51	2.16	0.93
2.0	—0.02	—0.02	—0.09	—0.05	0.61	2.19	2.85	2.28	0.23

TABLE 18.—Unit load distribution coefficients for example 3 $\theta = 1$, $\alpha = 1$

Beam Position	Load at $-b$	$-\frac{3b}{4}$	$-\frac{b}{2}$	$-\frac{b}{4}$	0	$\frac{b}{4}$	$\frac{b}{2}$	$\frac{3b}{4}$	b
0	0.475	0.650	0.935	1.350	1.625	1.350	0.935	0.650	0.475
$\frac{b}{4}$	0.280	0.275	0.570	0.895	1.350	1.690	1.454	1.100	0.888
$\frac{b}{2}$	0.140	0.219	0.330	0.570	0.935	1.450	1.850	1.760	1.560
$\frac{3b}{4}$	[illegible]	0.130	0.210	0.370	0.650	1.100	1.760	2.430	2.680

TABLE 19.—Combined distribution coefficients for example 3 $\theta = 1$, $\alpha = 1$

$-b$	$-\frac{3b}{4}$	$-\frac{b}{2}$	$-\frac{b}{4}$	0	$\frac{b}{4}$	$\frac{b}{2}$	$\frac{3b}{4}$	b
0.16	0.24	0.37	0.61	0.98	1.41	1.75	1.76	1.71

which failed by combined bending and torsion on the main beams.

11. Examples

As examples of the practical use of the methods the following sections show how it may be applied in three cases.

The first is the second simply supported test specimen, in which torsion did not occur, the second is the continuous specimen in which considerable torsional effects were

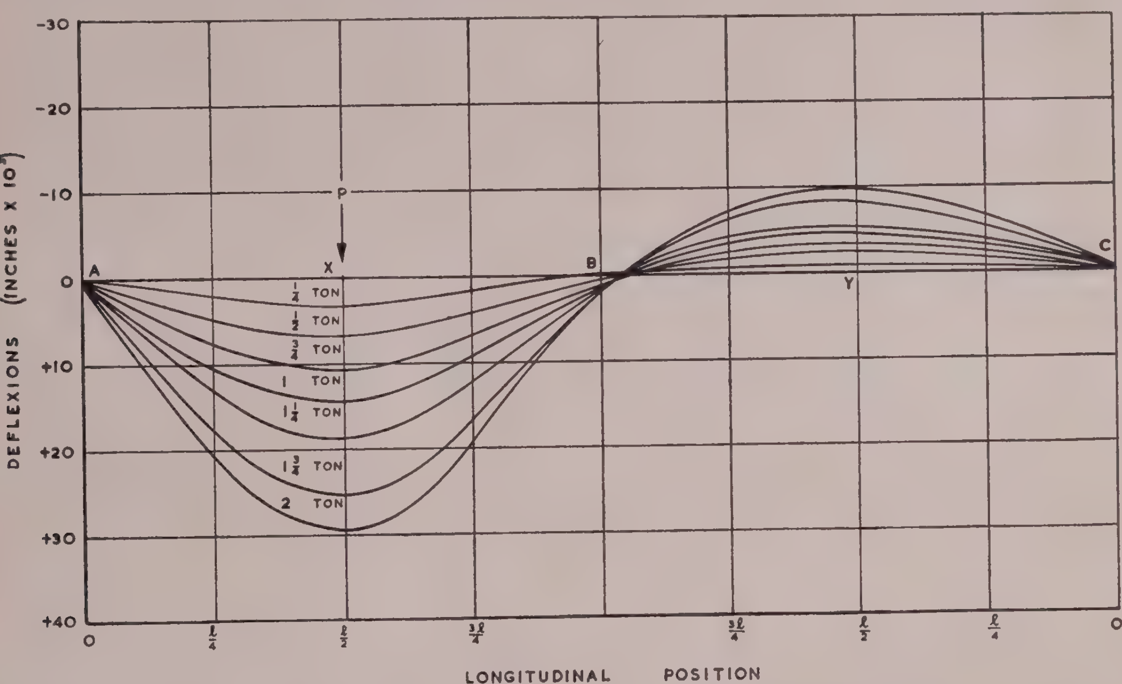


Fig. 29.—Measured mean deflexion profiles for the continuous specimen

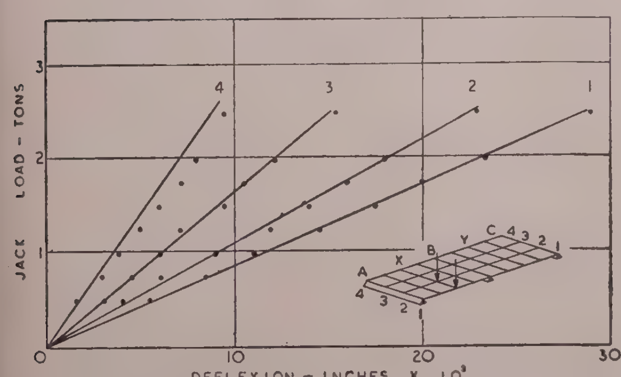


Fig. 30.—Comparison between the actual and the theoretical deflexions for the loaded span of the continuous specimen

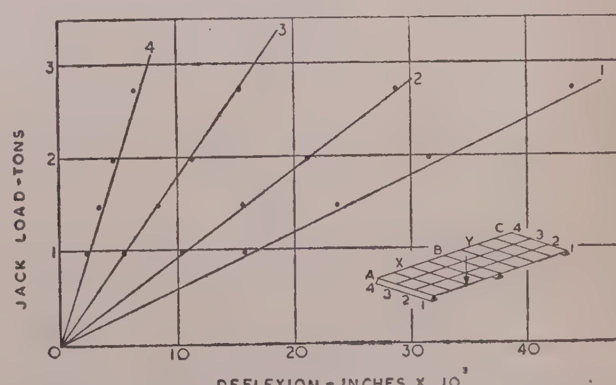


Fig. 31.—Comparison between the actual and the theoretical deflexions for the loaded span of the continuous specimen

noticed whilst the third is a suggested way of approaching an actual bridge structure. At present the authors are not satisfied with the information available on the torsional behaviour of Tee beam and box beam systems and, as has already been mentioned, tests are in progress on these forms of structure.

12. Example 1—The Second Simply Supported Specimen

The analysis of the "no-torsion" grillage No. 2 derives as follows :—

Data

No. of main beams	n	4
Dimensions of main beams		20 ft. 0 in. × 12 in. × 5 in.
No. of cross-beams	ν	3
Dimensions of cross-beams		22 in. × 7 in. × 3 in.
Spacing of main beams	p	33 in.
Actual width	S	8 ft. 3 in.
Effective width	$2b$	11 ft. 0 in.
$\frac{b}{2a}$		0.275
I		0.0347 ft.^{4}
$i = \frac{I}{p}$		0.01262 ft.^{4} per ft.
J		0.00413 ft.^{4}
$j = \frac{\nu J}{2a}$		0.00062 ft.^{4} per ft.
$\theta = \frac{b}{2a}\sqrt[4]{\frac{i}{j}}$		0.584

The intercepts of the line $\theta = 0.584$ on the Guyon curves of Figs. 4 to 9 give the unit load distribution Table 8. The values of Tables 8 are plotted in Fig. 36. The Table should be symmetrical about the marked

diagonals. The effective beam positions are again $-\frac{3b}{4}, -\frac{b}{4}, +\frac{b}{4}$ and $+\frac{3b}{4}$ and, hence, the final distribution Table for the load sequence of Fig. 20 is as given in Table 9.

Similarly the first and third specimens are defined by $\theta = 0.248$ and $\theta = 0.484$ respectively. The final distribution patterns are shown in Tables 10 and 11.

The "unit" distribution influence lines are given in Figs. 37 and 38 and can be compared with the curves for infinite transverse stiffness when $\theta = 0$ and $\alpha = 0$ of Fig. 39.

The "mean" deflexions and bending moments

Fig. 40 indicates the type of loading used in the test. The "mean" deflexion profile is given by

$$EIw = W\left[\frac{x^3}{6} - \frac{(x-d)^3}{6} - \frac{d}{2}(2a-d)\,x\right]$$

where

$$W = \frac{P}{\eta} = \frac{P}{4} \text{ (in this case.)}$$

The "mean" bending moments are given simply by

$$M = \frac{P}{\eta}x - \frac{P}{\eta}(x-d)$$

13. Example 2—The Analysis of the Continuous Grillage Specimen

Data

Overall length	13 ft. 4 in.	
Span	6 ft. 8 in.	$2a$
Actual width	2 ft. $5\frac{1}{4}$ in.	S
Beam spacing	$9\frac{3}{4}$ in.	p
Effective width $(p + S)$	3 ft. 3 in.	$2b$
Cross-beam spacing	1 ft. 8 in.	q
I	10.667 in.^{4}	
J	3.467 in.^{4}	

$$i = \frac{\eta I}{2b} = \frac{I}{p}$$

$$j = \frac{\nu J}{2a} = \frac{J}{q}$$

$$\frac{i}{j} = \frac{10.667}{9.75} \times \frac{20}{3.467} = 6.315$$

$$\theta = \frac{b}{2a}\sqrt[4]{\frac{i}{j}} = 0.387$$

TABLE 20.—Combined distribution coefficients for example 3 θ varying α = 1

θ	Beam at $-b$	$-\frac{3b}{4}$	$-\frac{b}{2}$	$-\frac{b}{4}$	0	$\frac{b}{4}$	$\frac{b}{2}$	$\frac{3b}{4}$	b
0	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
0.2	0.85	0.89	0.93	0.96	1.00	1.04	1.08	1.10	1.12
0.4	0.66	0.72	0.80	0.89	1.00	1.11	1.19	1.25	1.30
0.6	0.46	0.54	0.65	0.81	1.00	1.20	1.35	1.45	1.48
0.8	0.27	0.37	0.51	0.71	0.99	1.31	1.53	1.63	1.65

TABLE 21.—Tables of $(K_1 - K_0)$ values for varying values of θ

θ	$-b$	$-\frac{3b}{4}$	$-\frac{b}{2}$	$-\frac{b}{4}$	0	$\frac{b}{4}$	$\frac{b}{2}$	$\frac{3b}{4}$	b
0	1.5	1.12	0.75	0.38	0	—0.37	—0.74	—1.12	—1.5
0.2	1.34	1.02	0.68	0.34	0.01	—0.32	—0.65	—1.03	—1.37
0.4	1.15	0.85	0.51	0.28	0.01	—0.25	—0.56	—0.87	—1.15
0.6	0.96	0.68	0.44	0.20	—0.07	—0.23	—0.45	—0.65	—0.77
0.8	0.71	0.52	0.33	0.16	—0.05	—0.22	—0.38	—0.45	—0.49

TABLE 22.—Minimum values for α for a given θ to ensure that the working stresses are not exceeded

θ	Minimum value of α
0.2	0.57
0.4	0.76
0.6	1.09
0.8	2.38

Using the basic distribution coefficient curves for $\alpha = 0$ (Figs. 4 to 9) the Table of unit coefficients is constructed, Table 12.

These coefficients are the required coefficients since the effective load and beam eccentricities in the test specimen were $-\frac{3b}{4}, -\frac{b}{4}, +\frac{b}{4}$ and $+\frac{3b}{4}$ respectively.

TABLE 23.—Combined distribution coefficients for a grillage for which θ = 0.5 and α = 0.3

Beam Position	$-b$	$-\frac{3b}{4}$	$-\frac{b}{2}$	$-\frac{b}{4}$	0	$\frac{b}{4}$	$\frac{b}{2}$	$\frac{3b}{4}$	b
K	0.24	0.40	0.44	0.767	1.000	1.23	1.42	1.57	1.68

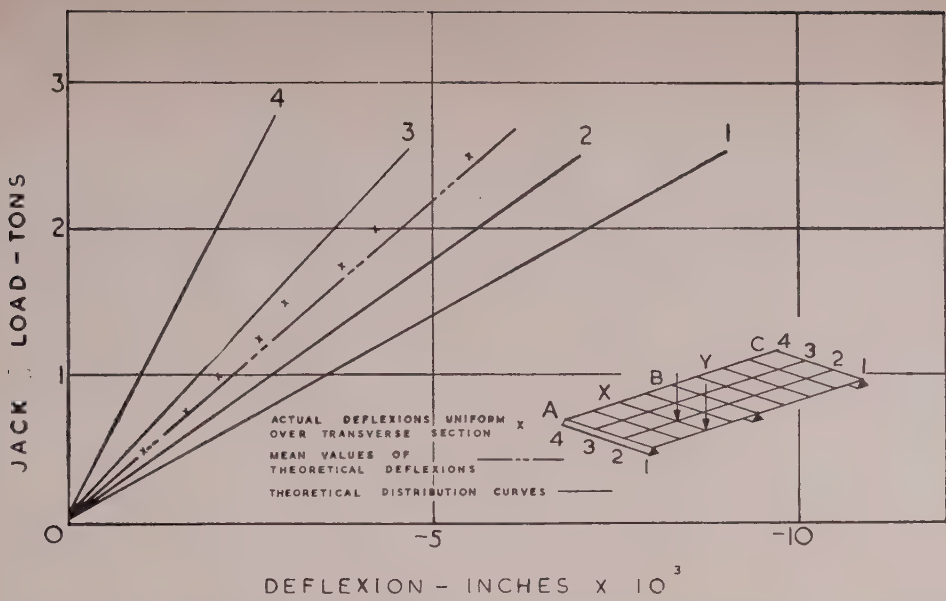


Fig. 32.—Comparison between the actual and the theoretical deflexions for the unloaded span of the continuous specimen

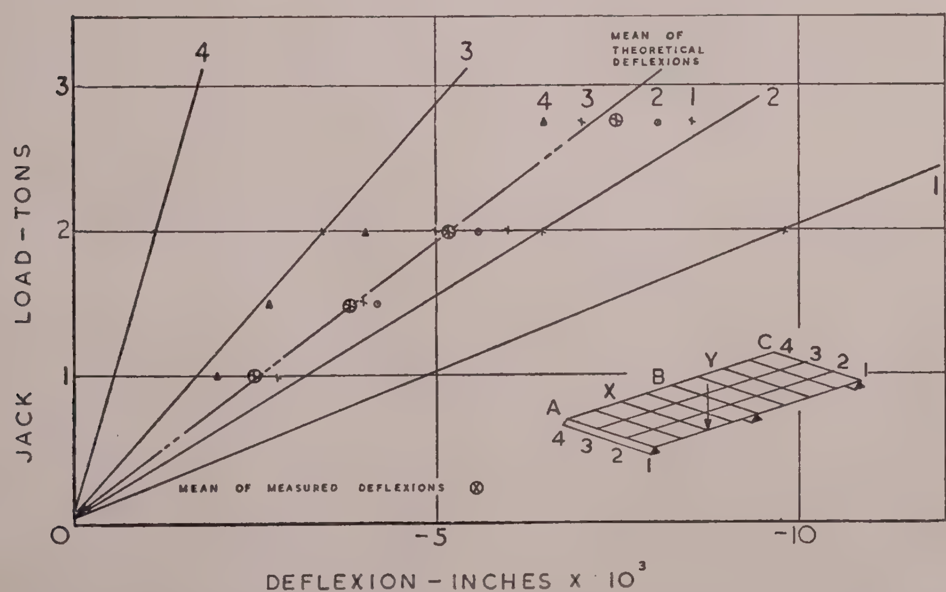


Fig. 33.—Comparison between the actual and the theoretical deflexions for the unloaded span of the continuous specimen

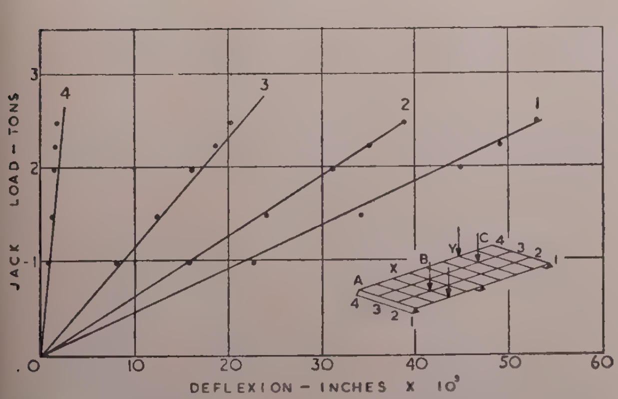


Fig. 34.—Comparison between the actual and the theoretical deflexions for two spans loaded of the continuous specimen

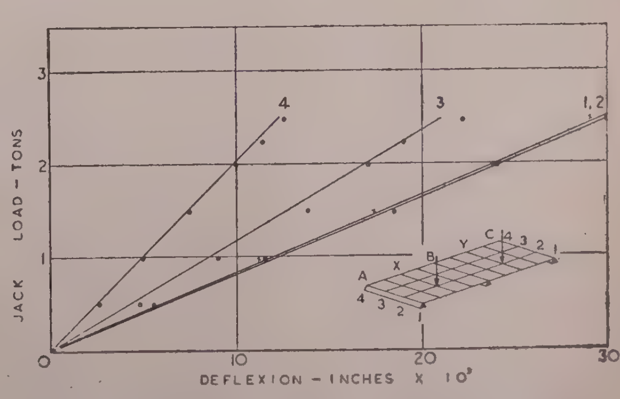


Fig. 35.—Comparison between the actual and the theoretical deflexions for two spans loaded of the continuous specimen

The resultant distribution for the loading positions shown in Fig. 20 are derived from Table 12 by superimposing the results for single loading. These resultant factors are given in Table 13.

The determination of the torsion parameter α

The analysis can be carried a stage further by taking into account the possible torsional restraint in the structure.

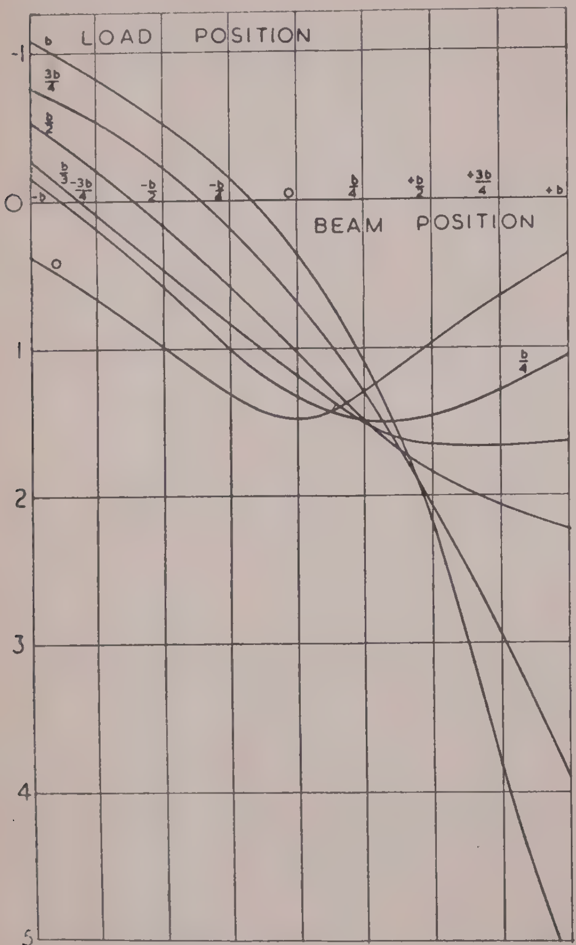


Fig. 36.—Influence lines for the distribution coefficients relevant to the second test specimen ($\theta = 0.584$)

The torsion is defined by the torsion parameter α where

$$\alpha = \frac{G\,(i_o + j_o)}{2E\,\sqrt{ij}}$$

Thus,

$$Ei = 1.0940\,E$$
$$Ej = 0.1734\,E$$

whence

$$\sqrt{ij} = 0.435$$

The general expression for the torsional rigidity N of a rectangular beam is by Timoshenko.

$$N = \zeta\,G\,d\,c^3$$

where c is the width of the beam, d is the depth of the beam and ζ is a factor whose value depends on the ratio $\frac{d}{c}$

For the main beams
$N_p = 0.229 \times 4 \times 2^3 \times G = 7.33\,G$
For the cross-beams
$N_E = 0.1835 \times 2.75 \times 2^3 \times G = 4.04\,G$

$$Gi_o = \frac{N_p}{p} = \frac{7.33\,G}{9.75} = 0.752\,G$$

$$Gj_o = \frac{N_E}{q} = \frac{4.04\,G}{20} = 0.202\,G$$

$$G(i_o + j_o) = 0.954\,G = 0.3816\,E$$

putting $G = 0.4\,E$

$$\alpha = \frac{G(i_o + j_o)}{2E\,\sqrt{ij}} = 0.438$$

The properties of the deck are, thus $\theta = 0.387$ and $\alpha = 0.438$.

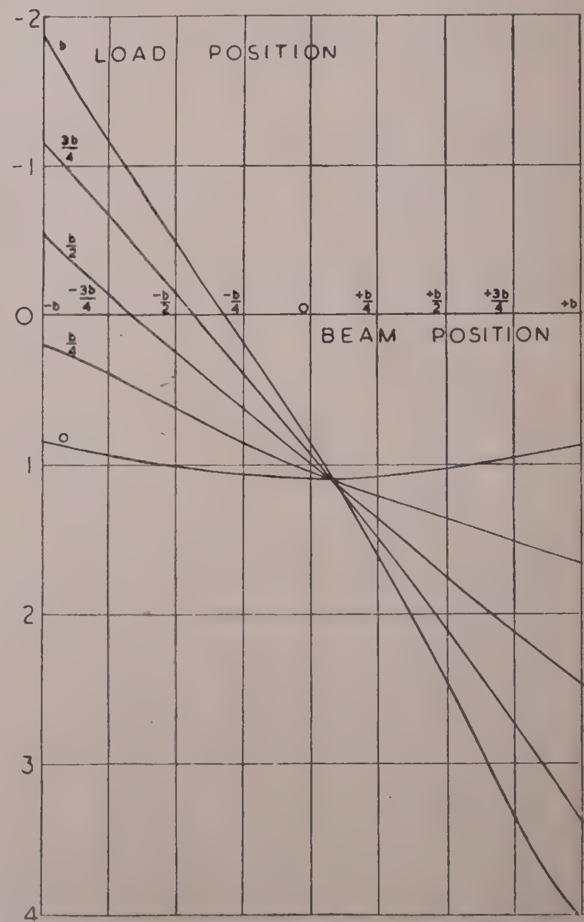


Fig. 37.—Influence lines for the distribution coefficients relevant to the first test specimen ($\theta = 0.293$)

Table 14 gives the values of K_1 derived from the full-torsion curves of Figs. 11 to 15.

The final distribution is obtained from Tables 12 and 14 from the relationship

$$K\alpha = K_o + (K_1 - K_o)\,\sqrt{\alpha}$$

and is given in Table 15.

Finally, for the load positions of Fig. 20, the resultant K values are as given in Table 16.

The "mean" deflexions

If we consider a continuous beam ABC with a concentrated load W applied at the centre of the span AB we have, by Clapeyron's Theorem of 3-moments

$$w = \frac{1}{EI}\left[\frac{Wx}{192}(13x^2 - 9l^2) - \frac{W}{6}\left(x - \frac{l}{2}\right)^3\right]$$

for the loaded span and

$$w = \frac{1}{EI}\left[\frac{Wl^2 x}{64} - \frac{Wx^3}{64}\right]$$

for the unloaded span.

The load W is equal to $\frac{P}{4}$ where P is the total load applied at the transverse section of the bridge.

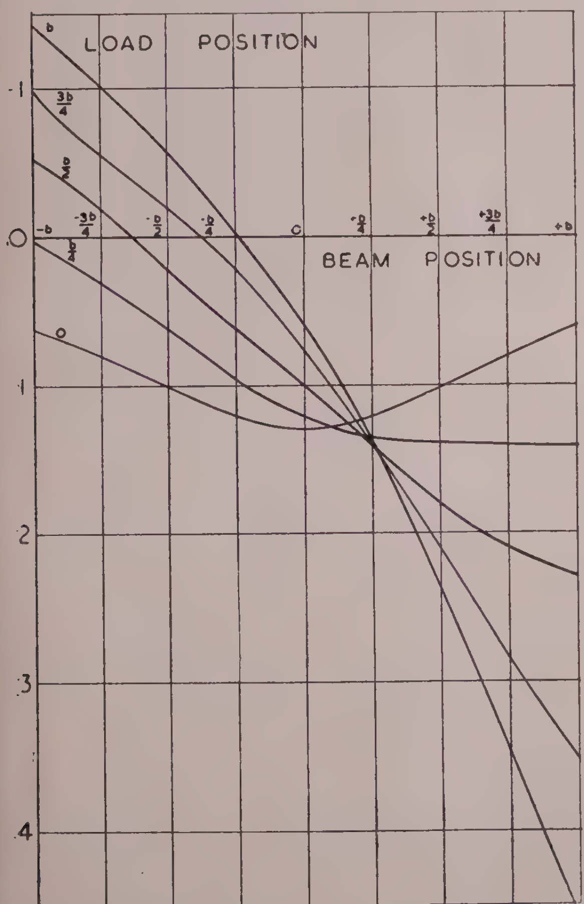


Fig. 38.—Influence lines for the distribution coefficients relevant to the third test specimen ($\theta = 0.484$)

If a settlement δ occurs at the mid-support we have for the unloaded span

$$w = \left[\left(\frac{\delta}{2l^3} - \frac{W}{64EI}\right)x^3 + \left(\frac{Wl^2}{64EI} - \frac{3\delta}{2l}\right)x\right]$$

and for the loaded span

$$w = \frac{1}{EI}\left[\frac{13}{192}Wx^3 - \frac{W}{6}\left(x - \frac{l}{2}\right)^3 - \frac{3Wl^2 x}{64} + EI\left(\frac{\delta x^3}{2l^3} - \frac{3\delta x}{2l}\right)\right]$$

These are the expressions which had to be used in comparison with the "measured" deflexion values.

The mean bending moments can be found in a similar manner.

The product of the "mean" effects and the coefficients of Table 16 gives the effects induced in the various beams for a given loading.

14. Example 3—A Beam Bridge Design

Consider a bridge with the following properties.

Data

Span	40 ft.	$2a$
Width	30 ft.	S
No. of main beams	7	η
Spacing of main beams	5 ft.	p
Effective width	35 ft.	$2b$
Working concrete stress	1,750 lb. per sq. in.	fc

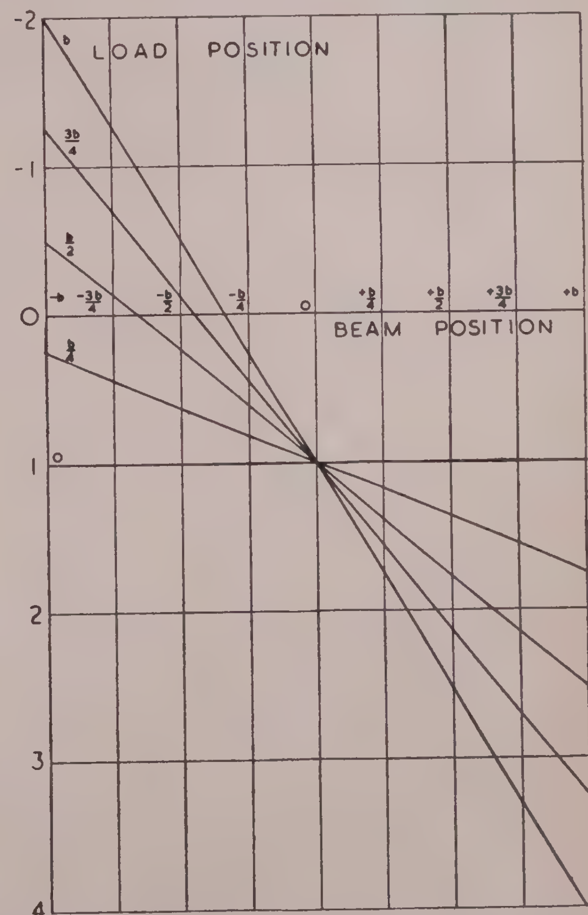


Fig. 39.—Influence lines for the distribution coefficients relevant to a bridge of infinite transverse stiffness ($\theta = 0$)

(*a*) *Normal Design for M.O.T. Standard Equivalent Loading*

U.D.L.	220 lb. per ft.
Knife edge load	2,700 lb. per ft. width
Maximum central bending moment	4,260,000 lb. in.

For a prestressed bridge assuming rectangular beams for simplicity the required modulus of each main beam is

$$Z = \frac{Mc}{fc} = \frac{4,260,000}{1,750}$$

$$\therefore \quad Z = 2,438 \text{ in.}^3$$
$$d = 31.25 \text{ in.}$$
$$\text{and} \quad b = 15 \text{ in.}$$

The required relaxed prestressing force is

$P = 410,000$ lb. per beam.

With an 8 in. thick decking slab included in the dead weight the total eccentricity of the prestress at mid-span becomes

$e = 11.0$ in.

(*b*) *Abnormal Loading*

If the abnormal load, shown in Fig. 42, is distributed uniformly between the seven beams of the deck there is a

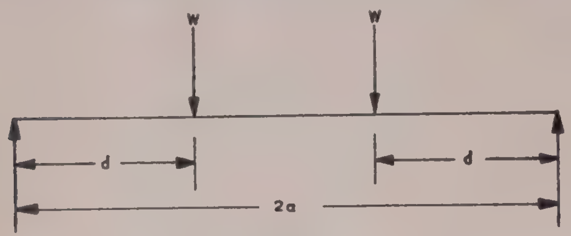


Fig. 40

train of four loads of $\frac{45}{7} = 6.43$ tons on each beam. The maximum moment caused by this train at the mid-section of each beam is

$M_c = 17\,W$

$= 2{,}940{,}000$ lb. in.

This is the " mean " moment for the deck. Thus, for a condition of zero stress at the lower face of the most heavily loaded beam, or beams, the maximum allowable distribution factor is

$$K_{max} = \frac{4{,}260{,}000}{2{,}940{,}000} = 1.445$$

A greater factor will result in a bending moment greater than the working moment.

The abnormal load has been placed on the deck such that the near side wheels correspond to the effective position $+\frac{3b}{4}$ as shown in Fig. 43. If the pavement is partially cantilevered this position could quite conceivably correspond to the boundary of the running surface.

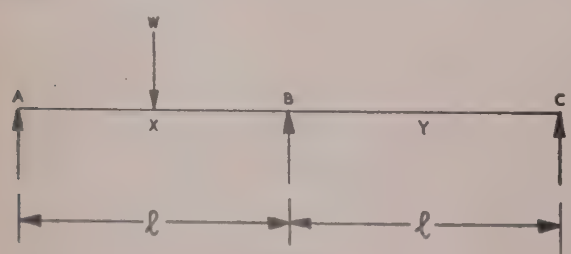


Fig. 41

The four loads are distributed to the effective positions and thus, the abnormal load is equivalent to equal loads of $1.333\,W$ at points $+\frac{b}{4}$, $+\frac{b}{2}$ and $+\frac{3b}{4}$ Fig. 43.

The combined " no-torsion " coefficients for this loading for various values of θ are given in Table 17.

The combined Table was deduced by superimposing the values for single loading at each of the three sections.

A value of $\theta = 1$ is indicated as the optimum value, giving the least value of K_{max}.

When $\theta = 1$. $K_{max} = 2.07$

The maximum sagging moment is then

$2{,}940{,}000 \times 2.07 = 6{,}090{,}000$ lb. in.

which is equivalent to a maximum induced bending stress of

$$\frac{6{,}090{,}000}{2{,}438} = 2{,}500 \text{ lb. per sq. in.}$$

The maximum hogging moment occurs in an outside beam where K has a value of approximately -0.2 which is equivalent to a bending stress of 242 lb. per sq. in.

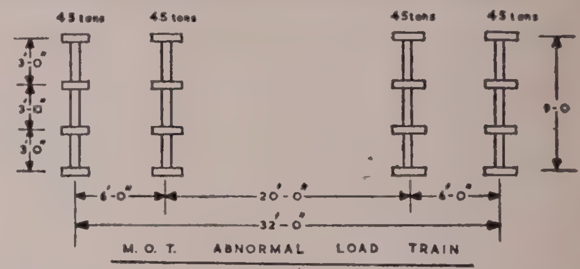


Fig. 42.—Ministry of Transport 180 tons abnormal loading

Thus, a tensile stress of 750 lb. per sq. in. is induced in the beam of eccentricity 10 ft. (as the values of K at the positions $\frac{b}{2}$ and $\frac{3b}{4}$ are practically identical). Note that although the actual beam positions have not been considered the K values for them can be interpolated

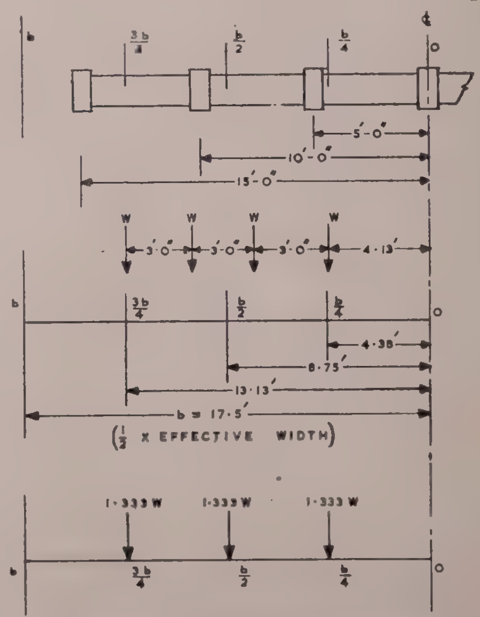


Fig. 43.—Abnormal loading points

from the values at the effective positions $\frac{b}{4}$, $\frac{b}{2}$, ... etc.

For $\theta = 1$, $i = 7{,}650$ in.^{4} $\frac{b}{2a} = 0.437$

Hence $j = 276$ in.^{4} per ft.

or $\frac{\nu J}{40} = 276$ in.^{4} per ft.

If $\nu = 9$, i.e., diaphragms at 5 ft. centres

$J \doteq 1{,}226$ in.^{4}

Thus, nine sets of diaphragms of section 14.25 in. $\times$ 5 in. would suffice in a " no-torsion " bridge.

If torsion is present we have for $\theta = 1$ and $\alpha = 1$ the full torsion coefficients of Table 18.

The combined coefficients for loads at $+\frac{b}{4}$, $+\frac{b}{2}$ and $+\frac{3b}{4}$ are, as in Table 19.

For a combined beam and slab construction the value of α will be less than unity and, hence, the maximum distribution coefficient will be greater than that given in the previous Table 19. In any case a bending moment in excess of the working moment would be induced by the abnormal load when $\theta = 1$.

In Table 20 the combined "torsion" coefficients for $\alpha = 1$ and various values of θ are given.

By relating Tables 17 and 20 the values of $(K_1 - K_0)$ are found as given in Table 21.

A maximum coefficient of 1.445 has been calculated previously if the working load is not to be exceeded.

Thus, for the relationship

$$K\alpha = K_0 + (K_1 - K_0)\sqrt{\alpha}$$

using Table 17 and 21 and putting $K\alpha = 1.445$ we find that the permissible values of α must not be less than the values given in Table 22.

A value of α exceeding unity is clearly impossible. Thus, the torsion deck must have a value of θ less than 0.5 if the working moment is never to be exceeded in any beam.

The procedure will be to dimension the components of the bridge system to satisfy the chosen value of θ. The value of α for the resultant system is then determined and checked against the minimum value. If this value of α is less than the minimum value as given by Table 22 the maximum value of $K\alpha$ must be re-determined from the relation

$$K\alpha = K_0 + (K_1 - K_0)\sqrt{\alpha}$$

Thus, the maximum tensile stress induced in the bridge by the abnormal loading is checked and the designer must decide whether or not such a stress is permissible.

As an example, if a simple rectangular beam grillage with torsional strength is considered then for $\theta = 0.5$

$$i = 7{,}650 \text{ in.}^4 \text{ per ft.} \qquad \frac{b}{2a} = 0.437$$

whence $j = 4{,}470 \text{ in.}^4$ per ft.

Thus, if the diaphragms are at 5 ft. spacing they must have a section 26.25 in. $\times$ 15 in. to give the required flexural rigidity

$$i_0 = \frac{1}{5} \times 0.23 \times 15^3 \times 31.25 = 4{,}825 \text{ in.}^4 \text{ per ft.}$$

$$j_0 = \frac{1}{5} \times 0.212 \times 15^3 \times 26.25 = 3{,}800 \text{ in.}^4 \text{ per ft.}$$

thus $$\alpha = \frac{G(i_0 + j_0)}{2E\sqrt{ij}} = 0.3$$

if $G = 0.4E$

The resultant coefficients for $\theta = 0.5$ and $\alpha = 0.3$, derived in exactly the same way as before, are given in Table 23.

The maximum coefficient occurs at an edge beam and is, by interpolation, equal to 1.63.

This value will cause a tensile stress of 225 lb. per sq. in.

This example illustrates that a value of α, appreciably less than the value required so that no tensile stresses are induced under full load, can still be acceptable due to the nature of the series which determine K.

Since the writing of this paper Dr. Janssonius has sent us a copy of his monograph[11] on relaxation methods applied to beam grillages and we feel that this is an important contribution to the relaxation method of approach.

Acknowledgement

The work described in the paper was carried out at the Research Station of the Cement and Concrete Association and is published with the permission of the Director.

Notation

Actual span	$2a$
Actual width	S
Effective width	$2b$
Longitudinal beam spacing	p
Cross beam spacing	q
Longitudinal beam 2nd moment of area	I
Cross beam 2nd moment of area	J
Longitudinal beam 2nd moment of area per unit effective width	i
Cross beam 2nd moment of area per unit length	j
Number of longitudinal beams	η
Number of cross beams	ν
Longitudinal co-ordinate	x
Transverse co-ordinate	y
Vertical deflexion	w
Longitudinal beam torsional moment of area	I_0
Cross beam torsional moment of area	J_0
Longitudinal beam torsional moment of area per unit effective width	i_0
Cross beam torsional moment of area per unit effective width	j_0
Young's modulus	E
Rigidity modulus	G
Bending moment	M
Distribution coefficient	K
Relative stiffness parameter	θ
Torsion parameter	α
Applied load	P
Load eccentricity	e
Beam eccentricity	f
Partial load term	r

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[10]Little, G. "Theories for interconnected bridge systems and their application to prestressed structures," M.Sc. Thesis, University of Durham, Newcastle-upon-Tyne, 1953.

[11]Janssonius, G. F. "Nieuwe Vereffeningsmethoden voor het Berelenen van Balkroosters." (New relaxation methods for calculation of grid frameworks.) Delft 1948. Thesis for the degree of doctor of engineering at the Technical Highschool at Delft.

Institution Notices and Proceedings

ORDINARY GENERAL MEETING

An Ordinary General Meeting of the Institution of Structural Engineers was held at 11, Upper Belgrave Street, London, S.W.1, on Thursday, January 28th, 1954, at 5.55 p.m., Lt.-Colonel R. F. Galbraith, M.C., B.Sc., M.I.Struct.E., A.M.I.C.E. (President), in the Chair.

The following members were elected in accordance with the Bye-Laws. Will members kindly note that the elections, as tabulated below, should be referred to when consulting the Year Book for evidence of membership.

STUDENTS

HALLIWELL, Alfred, of Farnworth, Lancs.
MARTIN, Ronald Victor, of Loughton, Essex.
STUBBS, Ian Rennie, of Stockport, Cheshire.

GRADUATES

AGGARWAL, Hari Dev. B.Sc.(Eng.) London, of Manchester.
BANNISTER, Geoffrey, B.Sc.(Tech.) Manchester, of Wad Medani, A.E. Sudan.
CORLETT, James Edward, of Saltburn by Sea, Yorks.
ENGLAND, Philip George, A.R.I.B.A., of Blackpool, Lancs.
FOX, Glyn Alan, of Trench, Nr. Wellington, Shrops.
HOGAN, Gerald, of Bury, Lancs.
JONES, Robin, of Kingswood, Bristol.
JORDAN, Edward John, of Warley, Smethwick, Staffs.
KEARNS, Lawrence, of Liverpool.
KERSHAW, Geoffrey Philip, of Stockport, Cheshire.
SARKAR, Santosh Kumar, of Knebworth, Herts.
TARAPOREVELA, Peshotan Jivanji, B.E.(Civil) Poona, of Bombay, India.

MEMBER

MINNIS, Brigadier Arnold, C.B.E., of Whitney on Wye, Hereford.

TRANSFERS

Students to Graduates

BRAND, Ronald Ernest, of London.
MASON, Peter Michael, of Stourbridge, Worcs.
NELSON, Kenneth, of Bolton, Lancs.
STANYON, Philip George, of Long Eaton, Notts.
WESTON, John, of Fearnhead, Nr. Warrington, Lancs.

Graduates to Associate-Members

JACKSON, James Keith, B.Sc.(Eng.) London, D.I.C., of Nottingham.
LEWIS, Frank Edgar, of London.

Associate-Members to Members

ELEY, George Frederick, A.M.I.C.E., of Allestree, Derby.
GEDDES, William George Nicholson, B.Sc.(Eng.), M.I.C.E., of Glasgow.
TALAAT, Mahmoud, B.Sc., Ph.D., of Giza, Egypt.

Members to Retired Members

RIGBY, Edward, of Southport, Lancs.

OBITUARY

The Council regret to announce the death of ARTHUR EAGLE and HAROLD FRANK PENTY (Members).

RESIGNATIONS

Notification was given that the Council had accepted with regret the resignations of JOHN RUTHERFORD OSMOND, CHARLES VINCENT RICHARDS (Members); WILLIAM STEPHEN BENTON (Retired Member); EDWARD DAVID BACHELOR RUSSELL (Associate); CLAUDE VIVIAN SMITH (Associate-Member); JOHN MARKHAM LEE HORSBURGH, JAMEEL NAZIH TALEB (Graduates); ALEXANDER HENRY WILLIS (Student).

EXAMINATIONS—JULY, 1954

The Examinations of the Institution will next be held at centres in the United Kingdom and overseas on July 13th and 14th, 1954 (Graduateship) and 15th and 16th (Associate-Membership).

RESEARCH AWARD

The Bronze Research Medal for the Session 1952-53 has been awarded to Dr. G. G. Meyerhof, for a paper on "Some Recent Foundation Research and its Application to Design" which was published in the Journal in June, 1953.

REPRESENTATION

The Council have renominated Mr. F. R. Bullen as the Institution's Representative on the Architects Registration Council and Admission Committee for the year ending March 1955.

FORTHCOMING MEETINGS

The following meetings will be held at 11, Upper Belgrave Street, London, S.W.1.

Thursday, March 11th, 1954

Ordinary Meeting, 6 p.m., when Mr. S. J. Crispin, M.I.Struct.E., L.R.I.B.A., will give a paper on "Soil Stabilisation in Fine Materials."

Thursday, March 25th, 1954

Ordinary General Meeting for the election of members, 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when Dr. P. B. Morice, B.Sc., and Mr. G. Little, M.Sc., will give a paper on "Load Distribution in Prestressed Concrete Bridge Systems."

Thursday, April 22nd, 1954

Ordinary General Meeting 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when Dr. R. Weck will give a paper on "Fatigue of Welded Structures."

Thursday, May 27th, 1954

Ordinary General Meeting for the election of members.
Annual General Meeting of the Institution.
Annual General Meeting of the Voting Contributors to the Institution of Structural Engineers' Benevolent Fund.

Members wishing to bring guests to the Ordinary Meetings announced above are requested to apply to the Secretary for tickets of admission.

SUMMER MEETING

As announced in the February issue, the Summer Meeting of the Institution will be held in Birmingham on May 18th-21st. Details of the programme have been circulated. Members who have not received these may obtain copies upon early application to the Secretary.

ANNUAL DINNER

The Annual Dinner will be held at the Dorchester Hotel, Park Lane, London, W., on Friday, March 26th, 1954, at 7 o'clock for 7.30 p.m. The Principal Guest will be the Rt. Hon. Lord Cherwell, C.H., F.R.S., Professor of Experimental Philosophy at the University of Oxford. The speeches will conclude at approximately 9.30 p.m. and will be followed by dancing until 2 a.m. Particulars have been circulated to members.

HONOURS AWARD

In offering their sincere congratulations to the following member on the distinction recently conferred upon

him, the Council feel they are also expressing the good wishes of the Institution.

COMPANION OF THE MOST DISTINGUISHED ORDER OF ST. MICHAEL AND ST. GEORGE

Mr. N. Wynne-Jones, C.B.E. (Member).

SESSIONAL PROGRAMME

The Literature Committee have under consideration the selection of papers for inclusion in the Sessional Programme for 1954-55. Members who may wish to offer papers during the coming Session are invited to communicate with the Secretary. The Committee would be glad to receive offers of papers on (i) Waterproofing and (ii) Water-sealing joints in concrete work.

MACLACHLAN LECTURE COMPETITION

The closing date for the receipt of entries for the MacLachlan Lecture Competition is Wednesday, March 31st, 1954. The general conditions of the competition are as follows :

1. The Institution of Structural Engineers shall institute a written lecture to be known as the MacLachlan Lecture and to be held annually.
2. The subject of the Lecture may be on any aspect of Structural Engineering as long as in every second year the subject shall be confined to steel structures.
3. Entrance into the competition for the Lecture shall be confined to Associate Members of the Institution, who are under the age of 32 years.
4. All papers entered for the competition shall be submitted to assessors to be appointed by the Council of the Institution, and all such papers (including the prize-winning Lecture) shall be available for publication in the Journal of the Institution at the discretion of the Council.
5. No paper submitted shall have been published or read elsewhere.
6. The winner of the competition shall be required to present the Lecture to a meeting of the Institution at which he will be presented with the sum of £17 10s.
7. Should a competitor's paper be considered worthy of ranking second in merit he will receive a consolation award of £5.
8. In the event of there being no winner of the competition in any one or more years, whether because no lecture submitted is considered to be of sufficient merit to warrant award, or for any other reason, the Institution shall transfer the above sums to the Research Fund of the Institution.

Particulars of the Competition for 1954

1. The MacLachlan Lecture will be given at a meeting of the Institution to be arranged towards the end of 1954.
2. The subject of the Lecture shall be on any aspect of structural engineering.
3. The work should be submitted as the script of a lecture which the author, if successful in the competition, will deliver before an audience in the course of about one hour. The development of mathematical formulæ and detailed calculations should be avoided as far as possible in the text ; if they are essential they should be embodied in appendices. Photographs, drawings, graphs, etc., which would appear as illustrations to the lecture in published form, should accompany the script. If additional illustrations would be shown as slides, a list of these should be included.
4. Six copies of each Lecture should be submitted and should be addressed to the Secretary of the Institution.
5. The closing date for the receipt of entries by the Institution is Wednesday, March 31st, 1954.

JOURNAL CASES AND BINDING, 1953

A binding case can be supplied for the twelve issues of the Journal, January-December, 1953 (Volume 31), price 11s. 6d., post free. The price for binding volumes is 27s. per volume, inclusive. This is for the half-leather binding which has been in use for some years.

It is requested that all parcels and Journals forwarded for binding should bear the name, address and rank of the member concerned. All volumes for binding must be despatched to the Institution by March 31st, 1954.

An Index will be included in all volumes bound. This Index will not be generally distributed, but members and others wishing to have a copy should apply to the Secretary.

PERSONAL

Lt.-Colonel F. D. Ogden, R.E. (Associate-Member) has been appointed Deputy Director of Works to the Prison Commission.

PRESIDENTIAL ADDRESS—AMENDMENT

January JOURNAL, page 29. The acknowledgement in connection with Fig. 49 should read "Sommerfelds Ltd." and *not* "R. A. Sefton Jenkins."

LONDON GRADUATES' AND STUDENTS' SECTION

The Annual General Meeting of the Section will be held at 11, Upper Belgrave Street, London, S.W.1, on Tuesday, March 9th, 1954, and will be followed by the showing of the film "The Failure of the Tacoma Narrows Suspension Bridge."

At the Annual General Meeting, a new Committee will be elected and it is hoped that as many members as possible will attend and make suggestions for the coming Session.

Hon. Secretary : J. F. S. Pryke, B.A.(Hons.), Bushcroft, Slipe Lane, Wormley, Herts

BRANCH NOTICES

LANCASHIRE AND CHESHIRE BRANCH

The following meetings have been arranged :—

Wednesday, March 10th, 1954

Joint Meeting with the Liverpool Engineering Society. "The Uses of Aluminium for Structural Purposes," by a member of the staff of the Aluminium Development Association. At the Temple, 24, Dale Street, Liverpool, 6 p.m.

Thursday, March 25th, 1954

Joint Meeting with the Reinforced Concrete Association, North-Western Branch. Mr. G. P. Bridges, M.I.Struct.E., A.M.I.C.E., L.R.I.B.A., on "The Design and Construction of Reinforced Concrete Silos and Bunkers." At the College of Technology, Manchester.

Wednesday, April 28th, 1954

Annual Business Meeting, followed by a film show.

All meetings, unless otherwise stated, will be held in the Reynolds Hall, College of Technology, Manchester, at 6.30 p.m., preceded by tea at 5.45 p.m.

Joint Hon. Secretaries : A. S. Sinclair, A.M.I.Struct.E., 17, The Circuit, Cheadle Hulme, Cheshire; M. D. Woods, A.M.I.Struct.E., 58, Spring Gardens, Salford, 6.

MIDLAND COUNTIES BRANCH

The following meetings have been arranged :—

Friday, March 26th, 1954

Mr. H. V. Hill, M.Sc., A.M.I.C.E., (Associate-Member of Council) on "The Load-Bearing Capacity of Metal Structures."

Tuesday, April 13th, 1954

Mr. Donovan H. Lee, B.Sc.(Eng.), M.I.C.E., M.I.Mech.E. (Member of Council) on "Recent Developments in Prestressed Concrete." At the Gas Showrooms, Nottingham, 7 p.m. The Secretary of the Institution will attend the meeting.

Friday, April 30th, 1954

Annual General Meeting, followed by a paper on "Some Factory Building Maintenance Problems," by Mr. W. T. Dudley, A.M.I.Struct.E.

Unless otherwise stated, meetings will be held at the James Watt Memorial Institute, Great Charles Street, Birmingham, at 6 p.m.

Hon. Secretary : L. A. Firminger, A.M.I.Struct.E., 656, Chester Road, Erdington, Birmingham, 23.

GRADUATES' AND STUDENTS' SECTION

Wednesday, March 31st, 1954

Address by the Chairman of the Midland Counties Branch, followed by the Annual General Meeting, at the James Watt Memorial Institute, Birmingham, 7 p.m.

Hon. Secretary : H. L. Bramwell (Graduate), 139, Wood Lane, Handsworth Wood, Birmingham, 20.

NORTHERN COUNTIES BRANCH

The following meetings have been arranged :—

Tuesday, March 2nd, 1954

Professor W. Fisher Cassie, M.Sc., Ph.D., F.R.S.E., M.I.C.E., M.I.Struct.E., on "Pavement Structures," at Middlesbrough.

Wednesday, March 3rd, 1954

The above meeting will be repeated at Newcastle.

Thursday, March 18th, 1954

Ladies' Guest Night, at Middlesbrough.

Friday, March 19th, 1954

Ladies' Guest Night, at Newcastle.

Wednesday, April 7th, 1954

Annual General Meeting, at Newcastle.

All meetings will commence at 6.30 p.m., the Middlesbrough meetings being held at the Cleveland Scientific and Technical Institution, Corporation Road, and the Newcastle meetings in the Neville Hall, near the Central Station.

Hon. Secretary : O. Lithgow, A.M.I.Struct.E., 4, Stoneleigh Avenue, Acklam, Middlesbrough.

NORTHERN IRELAND BRANCH

Tuesday, March 2nd, 1954

Film Evening.

The Annual General Meeting of the Branch will be held at the College of Technology, Belfast, on Tuesday, April 6th, 1954. The meeting will commence at 6.45 p.m. and will be preceded by tea at the Overseas League premises, Wellington Place, Belfast, at 6 p.m.

Hon. Secretary : A. H. K. Roberts, B.A., B.A.I., M.I.C.E.I., M.I.Struct.E., "Barbizon," 26, Dunlambert Park, Belfast.

SCOTTISH BRANCH

The following meetings have been arranged :—

Tuesday, March 16th, 1954

At the Ca'doro Restaurant, Glasgow, 6 p.m., Mr. F. R. Bullen, B.Sc., M.I.C.E. (Hon. Curator) on "Unusual Design for a large Constructional Shop."

Tuesday, April 13th, 1954

Annual General Meeting. At Ca'doro Restaurant, Glasgow, 6 p.m.

Hon. Secretary : G. Drysdale, A.M.I.Struct.E., "Niaroo," 33, Union Street, Motherwell, Lanarkshire.

SOUTH WESTERN COUNTIES BRANCH

The following meetings have been arranged :—

Friday, March 19th, 1954

Film on "Bridging," at the Demonstration Theatre of the South-Western Gas Board, Union Street, Torquay, 7 p.m.

Friday, May 21st, 1954

Annual General Meeting, at the Duke of Cornwall Hotel, Plymouth, 7 p.m.

Joint Hon. Secretaries : E. W. Howells, M.I.Struct.E., 10-12, Market Street, Torquay, Devon; C. J. Woodrow, "Elstow," Hartley Park Villas, Tavistock Road, Plymouth, Devon.

WALES AND MONMOUTHSHIRE BRANCH

The following meetings have been arranged :—

Friday, March 5th, 1954

At Swansea. Annual Dinner. The President and the Secretary of the Institution will be present.

Tuesday, March 9th, 1954

At Cardiff. Joint Meeting with the Institution of Civil Engineers. "Barry Dry-Dock Reconstruction."

Wednesday, March 31st, 1954

At Swansea. Mr. S. Woolf on "Recent Developments in Timber Structures."

Tuesday, May 11th, 1954

At Cardiff. Annual General Meeting.

Meetings at Cardiff will be held at the South Wales Institute of Engineers, Park Place, at 6.30 p.m.

Meetings at Swansea will be held at the Mackworth Hotel, at 6.30 p.m.

Hon. Secretary : G. R. Brueton, A.M.I.C.E., A.M.I.Struct.E., 86, The Exchange, Mount Stuart Square, Cardiff.

WESTERN COUNTIES BRANCH

The following meetings have been arranged :—

Thursday, March 4th, 1954

Combined Meeting with the Institution of Civil Engineers. Details to be announced.

Friday, April 2nd, 1954

Annual General Meeting, followed by a Film Show.

Unless otherwise stated, all meetings will be held at the University of Bristol Geology Lecture Theatre, at 6 p.m., preceded by tea at 5.30 p.m.

Hon. Secretary : E. Hughes, A.M.I.Struct.E., 23, Southdown Road, Westbury-on-Trym, Bristol, 9.

YORKSHIRE BRANCH

The following meetings have been arranged :—

Friday, March 12th, 1954

Annual Dinner and Dance, at Parkway Hotel, Bramhope, Leeds, 7 p.m.

Friday, March 26th, 1954

Joint Meeting with the Yorkshire Association of the Institution of Civil Engineers, in Hull. Professor A. L. L. Baker, B.Sc., M.I.C.E., M.I.Struct.E. (Vice-President) on "Jetties and Fenders."

Wednesday, April 28th, 1954

Annual General Meeting, followed by a paper by Dr. J. L. Murdoch, M.Sc., on "The Quality Control of Concrete."

All meetings will be held at the Great Northern Hotel, Leeds, at 6.30 p.m., except where otherwise stated.

Hon. Secretary : E. Wrigley, A.M.I.Struct.E., 17, The Drive, Alwoodley, Leeds.

UNION OF SOUTH AFRICA BRANCH

Branch Hon. Secretary : A. E. Tait, B.Sc., A.M.I.C.E., A.M.I.Struct.E., P.O. Box 3306, Johannesburg. During week-days Mr. Tait can be contacted in the City Engineer's Department, Town Hall, Johannesburg. 'Phone: 34-1111, Ext. 257.

Natal Section Hon. Secretary : E. G. Bennett, A.M.I.Struct.E., c/o The Reinforcing Steel Co., Ltd., P.O. Box 49, Merebank, Durban.

Cape Section, Hon. Secretary: R. Stubbs, M.I.Struct.E., The Reinforcing Steel Co., Ltd., P.O. Box 2962, Cape Town.

Fatigue of Welded Structures*

By R. Weck, Ph.D., A.M.I.C.E., A.M.I.Mech.E.

CONTENTS

FATIGUE FAILURE IN STRUCTURES

Appearance of Fatigue Failure

The structural engineer has abandoned cast iron as a constructional material largely because of its lack of ductility in the tensile test and demands nowadays that all metals used for load carrying parts should possess appreciable ductility. Ductility is of course an essential requirement for fabrication but with the use of ductile metals has grown up the idea that a structural part will exhibit appreciable deformation and so give warning before it fails. This is true enough if failure is due to an overload once accidentally applied but large numbers of failures in service occur every year which give no such warning, are not preceded by any visible deformation even in materials with 20 per cent. or more percentage elongation, and are not caused by loads in excess of those for which the part has been designed. The fracture appears as a crack, that is a clean break, and its surface is generally of smooth, velvety appearance. An example of two such cracks in one welded beam is shown in Fig. 1. All such failures are caused by a very large number of load applications, often all of them in the range of permissible stresses. The phenomenon is known as "Failure from Fatigue."

The structural engineer is relatively fortunate in that few of his creations are subject to a large enough number of stress variations of sufficient magnitude and frequency to induce fatigue failure, unlike the mechanical engineer for whom the risk of fatigue failure is ever-present in virtually everything he deals with.

Buildings

The risk of fatigue failure is entirely absent in buildings unless there are travelling cranes. Even then the number of times the crane is used to full capacity and travels over any appreciable distance of the gantry may not be large enough during the probable lifetime of the structure to result in a sufficiently large number of loading cycles to affect the gantry itself which would in any case be the only part of the building thus affected. Wind loading is variable and produces stress fluctuations but neither their number nor their magnitude is large enough for fatigue failure to result.

Brief reference must be made to one particular question which is sometimes posed, particularly in connection with welded building structures : the possibility of fatigue failure in consequence of vibrations produced by machinery. The problem was included in the work of the Steel Structures Research committee in the thirties and very elegantly answered by A. J. Newport[1]. He showed that "vibrations will become intolerable to the occupants of the room long before there is any danger of fatigue failure at the welds, with the exception of very low frequency large amplitude vibrations, in which case the obvious movement and uncomfortable vibration would probably call attention to the danger."

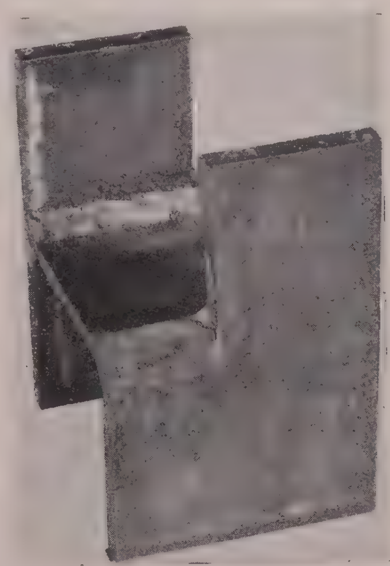

Fig. 1.—Two fatigue cracks in welded H-beam. One starting from stray flash, the other from accidental hammer mark

Fifteen years have passed and this conclusion has been fully borne out by experience. as far as is known. No fatigue failures caused by vibrating machinery have been reported in welded structures.

There may be special types of unoccupied structures where appreciable vibrations or repeated shocks are introduced intentionally such as in the structures of electrostatic precipitators. In such cases the stresses due to the vibrations should be ascertained and compared with those which can be safely sustained a very large number of times.

Whereas variations in wind force by themselves will not produce fatigue failure, wind may lead to fatigue failure if certain parts of a structure are excited to resonance vibrations by the action of the wind. The

Paper to be read before the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1, on Thursday, April 22nd, 1954, at 6 p.m.

only remedy in such cases is to change the aerodynamic characteristics of the member or its natural frequency.

Bridges

In highway bridges, those stresses due to live load which occur a very large number of times are generally only fractions of the full design stress and in general will not result in fatigue failure—although this depends largely on detail design—whereas the full design stress is not reached frequently enough during the life of the bridge to cause trouble.

The situation in railway bridges is very different. In most railway bridges the full live load stress may be reached in the main girders, every time a train passes over it and the cross girders may experience their full design stress many times even during the passage of a single train. There may nevertheless be large numbers of railway bridges, particularly in remote parts of the country and in overseas territories where the traffic density is so small that even in a century the number of stress cycles experienced is too small to cause trouble.

Machinery Structures

Buildings, highway- and railway-bridges cover the bulk of the structural engineer's work and it is only in structures forming part of a machine—apart from railway bridges and crane gantries—where fatigue becomes important to the structural engineer. The structures of all types of cranes, the frame structures for presses, the beams and turrets of excavators and draglines, the structures of all types of road and rail vehicles, and of course aircraft structures, are exposed to failure from fatigue. It is in this borderland between structural and mechanical engineering where fatigue becomes most important. More important perhaps than in mechanical design proper where, on the basis of experience extending over many years, crankshafts, connecting rods, gear wheels and other machine parts are designed for stresses which are much lower than those used by the structural engineer. The methods of design and analysis for machine parts do not fundamentally differ from those used by the structural engineer, but by using low values of permissible stresses account is taken implicitly of fatigue loading.

In the design of structures, even if they are part of a machine, the conventional, well-tried methods of structural design are used which assume that the loads act statically. Even if different cases of loading are considered, each case is considered separately and the fact that there may be periodic or continuous variations between different cases of loading is ignored. A concession to fatigue loading—if it is not ignored altogether—is made by some reduction in permissible stresses but otherwise a dragline boom is designed in the same way as say a transmission tower or a roof truss. In order to understand that design methods which are perfectly adequate for building structures, highway bridges transmission towers and roof structures are not entirely adequate for mechanical structures certain fundamental facts of the fatigue phenomenon have to be considered. Design for fatigue loading really requires a different approach.

Before considering some of the fundamental facts about fatigue which must influence the designer, it may be useful to make some general observations.

General Aspects of Fatigue Failure in Structures

Since fatigue failure will occur only as the result of a large number of loading cycles, it is inevitable that it may not develop for several years and in some cases very many years. From the point of view of the designer and manufacturer this is a mixed blessing. Their interest in and responsibility for the structure may have become somewhat remote by that time unless, as is the case with railways, the designers are at the same time responsible for maintenance. Unless fatigue failure develops fairly early in the life of a structure neither the designer nor the manufacturer, least of all the larger engineering community will hear of it and this may have created the impression that fatigue failures in structures are rare. This is a mistaken impression. It requires much effort to obtain information on failures in service but once one starts digging one finds that far more fatigue failures occur than is generally supposed.

In railway bridges they are dealt with in the ordinary course of maintenance and accepted as an inevitable evil scarcely worth recording. This is a great pity, because only from careful study and interpretation of service experience which constitute the only full-scale tests we can afford can our knowledge be increased and our practice be improved. Laboratory tests can in the best circumstances be a guide requiring confirmation by the full-scale test in service.

The fact that fatigue failures do not become evident, sometimes for many years after the structure has been commissioned, has the serious consequence that many similar structures incorporating the same shortcomings may have been constructed in the intervening period which may all sooner or later fail in the same manner. Since it takes so long for the result of the full-scale experiment to become known laboratory experiments imperfect as they are, are indispensable in extending our experience.

The engineer associates with " failure " a state of the structure when it collapses or becomes entirely un serviceable. Fatigue failure rarely takes this catas trophic form. To quote an example from anothe branch of engineering, approximately thirty fatigu failures in civil aircraft are reported to the Civil Aero nautics Authority in the United States every month Of these, serious failures of the airframe resulting in crash are extremely rare, though they do of course occur

The structural engineer may require to adjust hi outlook on what constitutes failure. The dictum tha the designer must make certain that failure—any typ of failure—does not occur and which finds unquestionin and axiomatic acceptance amongst structural engineer may have to be abandoned or at least modified wit regard to fatigue failure. For one thing, absolut freedom from fatigue failure cannot be assured becaus fatigue is now recognised to be a statistical phenomenor All that can be expected of the designer is to mak certain that the incidence of fatigue failure does no exceed a certain economically tolerable and reasonabl safe minimum.

In deciding to what extent fatigue should be take into account in the design of a particular structure, th possible consequence of such failure after many years i service must be assessed. Two cases must clearly b distinguished, those where fatigue failure might lead t loss of life, serious and costly disorganisation of pro duction or transport or other heavy material loss, an cases where there may be some material loss but whe human lives would not be endangered and where fatig damage may be repaired—as other accidental dama is—at relatively modest cost. It must be accepted as fact that absolute safety against fatigue failure cannot b achieved even if cost is of no account. Cost scarce counts in aircraft structures and yet fatigue failures c occur. The whole fatigue problem must be considere from the point of view of cost *versus* safety becau increased safety against fatigue damage can general

be obtained only at appreciably greater constructional cost, provided of course that the more obvious mistakes which may result in fatigue failure and which it is possible to avoid without greater cost are avoided in any case.

In deciding this question not only the purpose but the type of structure are important. The more highly redundant the structure is the less serious will be the consequences of a fatigue failure. If a fracture developed in the region of maximum bending moment in a crane girder continuous over several supports the chances are that the girder could continue to function whereas the same fracture in one of a number of single spans might lead to a very serious accident.

Structural engineers, by virtue of the fact that most of their work is not subject to fatigue loading, have been able very largely to ignore fatigue altogether even in such structures as railway bridges which are subject to fatigue loading. They have undoubtedly benefited from this fact that a fatigue failure in a structure is not always catastrophical and can be repaired. For those structures where large numbers of loading cycles are likely to be accumulated in a relatively short time and where a fatigue failure may produce the collapse of the structure and lead to loss of life or serious material loss the designer should consider fatigue as one of the critical design conditions and this requires in general a fairly drastic departure from standard practice. In conventional design the dimensions of the members are and can be fixed before even the method of joining them is decided. When fatigue is the criterion the method of joining, and in many cases the exact detail of the joints, must be decided before the sizes of the members can be determined. This is only one aspect of the fatigue problem as it affects our accustomed way of thinking. There are other aspects requiring some modification of the conventional approach. They all follow directly from certain very elementary facts concerning the fatigue of metals.

SOME FUNDAMENTAL FACTS OF FATIGUE OF METALS

The S.N. Diagram and Its Significance

Fatigue failure is produced by stresses very much smaller than the ultimate tensile strength of the material. Under certain circumstances, stresses above the yield point may be sustained millions of times without producing damage. In general, the higher the stresses applied the smaller will be the number of cycles to failure and *vice versa*. There is some idea that the permissible stresses used in practice do not exceed the elastic limit of the material and that fatigue need therefore not be considered. This idea is entirely false. In general, only average or nominal stresses are calculated and limited to permissible stress values. The stress concentrations are ignored and therefore fatigue failure may still occur and does frequently occur in structures whose nominal design stresses do not exceed the so-called elastic limit. In considering fatigue it is best to forget all about the static properties of materials.

The relation between stress and number of cycles to failure is given by a curve which at first drops steeply with increasing number of cycles to become increasingly flatter in the region of greater endurance until the stress reaches the so-called endurance limit which is the maximum stress that can be applied infinitely often without producing failure. There is some question whether an endurance limit does in fact exist but for practical purposes one is in any case not concerned with an endurance limit for infinite life, since no engineering product is expected to last for eternity, but with a limited life corresponding to the life to be expected of the structure. Stress value for definite limiting lives such as two million or ten million are therefore used in place of the endurance limit. For light alloys an endurance limit does in any case not exist but this fact in itself makes no difference to the use of light alloys in structures subjected to fatigue conditions.

There is one rather tricky question which is often raised, particularly by structural engineers : How large can the number of cycles be for a structure before fatigue has to be considered ? Can fatigue failure be produced by 100, 1,000, 10,000, 100,000 cycles ? Alternatively, can we forget all about fatigue as long as the number of stress cycles the structure has to endure during its life does not approach, say, the million mark ?

This question cannot be answered correctly in general terms. Undoubtedly for small numbers of cycles it is more appropriate to speak of " repeated loading " rather than fatigue. There is no definite dividing line between " repeated loading " and fatigue proper. To produce failure at less than 10,000 cycles will generally require—even if no care in the design of detail is exercised—nominal stresses well in the plastic range of the materials so that the repeated applications of the load would be accompanied by appreciable plastic deformation with a permanent set constantly increasing with every cycle. The fracture may have less the character and appearance of a fatigue failure than that of ordinary static failure. Generally, nominal stresses—as distinct from highly localised stress concentrations—of this magnitude are not likely to occur in structures.

Fatigue failure proper can certainly occur at 100,000 cycles and at stresses which do not exceed those generally considered permissible but only if there are design details somewhat susceptible to fatigue failure. As an extreme example, the joint in Fig. 2 may be considered.

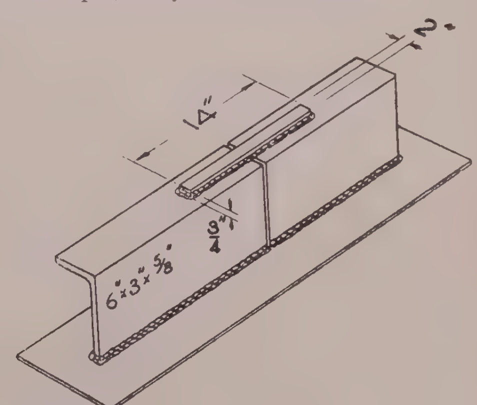

Fig. 2.—Joint of angle subject to alternating bending of ± 8 tons/in.^{2} Failure in strap after approximately 50,000 cycles

A joint between two unequal angles was effected by a plate welded to the toes of the longer leg and a relatively small strap was welded across the other leg. This joint when subjected to ± 8 tons/in.^{2} maximum bending stress started to crack after 46.620 cycles and the fatigue crack had passed through the strap after another 100,000 cycles.

Fatigue failure at stresses in the range of permissible stresses used at present can certainly occur between 100,000 and 1,000,000 cycles for standard details which—unlike the example just quoted—would be perfectly satisfactory for static loading and for stress variations no larger than those considered permissible for static loading.

Conversely, stresses very much smaller than those considered permissible may produce fatigue failure

after some millions of cycles if a detail is used which is particularly susceptible to fatigue failure. It does not follow that a particular member which will withstand 100,000 cycles at 10 tons/in.^{2} will withstand 2,000,000 or more at 5 tons/in.^{2} This fact raises a very difficult problem. The number of times a member is subjected to its full design stress in, say, a highway bridge, may be small but the number of times it is subject to some fraction of the maximum design stress may be very large. Fatigue failure may occur as much as a result of a relatively small number—say 500,000 of high stresses as of a very large number of relatively small stresses. Each stress level really requires to be checked separately. To do this would require not only complete stress analysis for a large number of practical loading conditions in addition to maximum loading together with the knowledge of the number of cycles each of these loading conditions are experienced. This information "the loading spectrum" is generally not ascertainable. Even if it were, it would be of little use because the "safe" stress level of any given detail is known generally only for one particular number of cycles, say, 2 million, and is not known for numbers of cycles smaller and larger than this. This however does not exhaust the complexity of the problem. Even if the safe stress level were known for a fairly large number of endurances so that the S-N diagram could be drawn for each detail of the bridge one is still faced with what is commonly known as the "cumulative damage" problem. It is basically the question of how many stress cycles can be safely endured at one given stress level when this is preceded and followed by more or less arbitrary number of cycles at several other stress levels.

Fortunately, we know from experience that highway bridges have remained largely immune from fatigue failure. The whole problem of fatigue in highway bridges could be safely ignored were it not for the fact that permissible stresses are gradually increasing and new constructional methods such as welding are introduced on which we cannot claim to have experience over a sufficiently large number of years.

The Notch Effect

Practically everything in the fatigue of structures depends on the notch effect, which is by far the most important cause of reductions in fatigue strength. It occupies a central and crucial position in all fatigue considerations. The immediate effect of a notch—and every discontinuity can be considered a notch in this sense—is a disturbance in the stress distribution. A single rivet hole in a tension member may produce local increases in average tensile stress in the member to anything up to three times the value of the average at two diametrically opposite points at the edge of the hole. Under static loading such stress concentration can be ignored but since fatigue is a purely local effect they become of overriding importance.

A fatigue crack always starts from the point of highest stress and gradually grows from microscopic size until it passes through the entire cross-section. In the perforated tension member the fatigue crack will start from the end points of the diameter at right-angles to the direction of the applied load where the stress may be nearly three times the average and it is this local stress value which determines the fatigue resistance of the whole member. If it be assumed that the fatigue strength of the unperforated member is F and the ratio of maximum stress Sp at the edge of the hole to the average stress in the member S is equal to a factor $k = \frac{Sp}{S}$ which is termed the stress concentration factor, and if it is further assumed that the stress Sp cannot exceed the fatigue strength F of the material, the average stress in the member cannot exceed $\frac{F}{k}$ without the risk of failure. If in the present example the fatigue strength of the unperforated member were, say, $F = 15$ tons/in.^{2} in tension, a single rivet hole with a stress concentration factor $k = 3$ would reduce the fatigue strength of the member to 5 tons/in.^{2} The cross-sectional area of the member would therefore have to be increased by a factor of three in order for the perforated member to develop the same fatigue strength as the unperforated member.

In actual fact the reduction in fatigue strength is not always equal to the stress concentration factor, at least not for mild steel. The reduction in fatigue strength produced by a single hole—which it must be remembered is a relatively mild form of notch when compared with the effect of other structural discontinuities—would not be quite as severe as the figures chosen to illustrate the point would indicate. For one thing the stress concentration factor for a single hole in a plate of finite width is less than 3 and even this does not become fully effective in a material such as mild steel. In steels of higher tensile strength the stress concentrations may become fully effective once the component exceeds a certain size as was demonstrated by Phillips and Heywood.[2].

If a single hole can affect the fatigue strength so drastically it might be thought that multiple perforations as required for riveting will have an even more deleterious effect. This is not the case. Provided the holes are suitably arranged the stress concentrations due to a number of holes may be less severe than those due to a single hole unless the holes are very close together. The stress concentrations from different holes reduce each other through interference. The same phenomenon is observed with bolts where the fatigue strength of the threaded portion is higher than that of a rod of the same diameter with a single thread or V-groove.

The fatigue strength of a member depends entirely on the stress concentrations produced by the discontinuities it contains. Since generally the most severe structural discontinuity occurs where one member is joined to others its fatigue strength—unless the stresses at the joint are negligible—will depend almost entirely on the form of the joints. Before the form of the joints in a member is settled in some detail it is impossible to say anything about the fatigue strength of the member and hence a safe permissible stress for the member, and hence its cross-section cannot be determined unless the form of the joint is fixed first.

Fatigue Limit of the Material

The structural engineer has got the idea to fix permissible stresses as some fraction of a property of the material, be it the yield point or the ultimate tensile strength. If this works for conditions of static loading why should it not work for fatigue loading as long as the "fatigue strength" of the materials is used as the mechanical property instead of the yield point of the material. The reason why it does not work is that the fatigue strength of the material is not a mechanical property in the same sense as, say, the yield point. If a tension member is designed for a permissible stress equal to half the yield point, that is with a safety factor of two against yielding, the member would not yield until a load is reached roughly equal to twice the design load even if the member contained a hole. If the tension member is designed for half the tensile fatigue

strength of the material for, say, 2 million cycles, it would certainly fail at well under 2 million cycles. This is because the stress limited to half the two million cycles limit is the nominal or average stress in the member and not the real maximum stress which might occur somewhere in a joint and which is normally not calculated, and probably not calculable. This stress will exceed by far the fatigue strength of the material.

Perhaps this can be illustrated with an example.[3] The fatigue strength of a particular rail steel of 50.6 tons/in.^{2} tensile strength is 33 tons/in.^{2} This fatigue strength is determined in the usual way by using small polished test pieces not unlike tensile test pieces. The fatigue strength of a rail made from this steel is 24 tons/in.^{2} If the rail is joined by flash welding, thermit, arc or gas welding the fatigue strength would be 21.6, 12.7, 12.7 and 11.4 tons/in.^{2} respectively. Obviously, if the rail was designed for half the fatigue strength of the material, i.e., 16.5 tons/in.^{2}, it would be all right if it contained no joints, or if the joints were made by flash welding, but it would be expected to fail at well under 2 million cycles if the joints were made by any other welding process. A rail joint made by bolting would produce similar reductions in fatigue strength.

The "fatigue strength" of a material depends very much more on the way in which it has been determined than any other mechanical property. It depends on the size, the surface condition, and the form of the test pieces used for its determination, and there is no clear relation between the fatigue strength determined by testing small test pieces of the metal and the fatigue strength realised in a full-scale member. This is well-illustrated by work carried out by Gaber in Karlsruhe.[4]

He carried out fatigue tests with full-size riveted bridge truss diagonals, including of course the end attachments at a stress of 7.7 tons/in.^{2}, which was the design stress used for many of the older riveted bridges in Germany. He found that only one out of twelve test pieces sustained more than one million cycles of the design stress, some failed after little more than 100,000 cycles.

The stress of 7.7 tons/in.^{2} is certainly less than half the fatigue strength of the steel used for these bridges. The fatigue strength of the material in fact bears no relation to the stress which can be safely sustained 2 million times by a riveted bridge diagonal.

It is interesting to note in this context that Gaber came to the conclusion that the absence of fatigue failure in these older bridges must be due to these facts :

1. The actual live load is smaller than the design load.
2. The impact factors used in calculations are too large.
3. The load position producing maximum design stress in the diagonals does not occur very frequently.
4. In consequence of the rigidity of connections and the presence of bracing, every bridge is a space frame in which the stresses are smaller than those calculated on the basis of the analysis for the main girders as separate plane structures.

Effect of Mean Stress

In most structural members stress variations do not necessarily take place between either equal tensile and compression stress limits or between zero and a maximum because the stress variations due to live load are superimposed on the constant stress due to dead load. Hence, the mean stress between upper and lower stress limit is different for different structures and for different members in the same structure. The fatigue strength is however dependent on the mean stress. The greatest fatigue range between minimum and maximum stress is obtained for zero mean stress and the safe range decreases for increasing positive or negative values of the mean stress.

This means in practice that the safe upper limit of periodic stress variation is increased somewhat by the presence of a steady stress as compared with variations between zero and a maximum but the increase is smaller than the steady stress, so that the total amplitude of variation is reduced. If there is reversal of stress the total range of stress will be increased as compared with variations between zero and a maximum, but both the tensile and the compressive limit will be reduced in comparison with the fatigue strength for variation between zero and, say, maximum tension or compression.

This relation is generally represented in a Smith diagram—see Fig. 3, where C″-C″ is the alternating fatigue strength, A″-A″ the basic tensile fatigue strength.

Corrosion

If metals are exposed to corrosive influences whilst subject to fatigue loading it is found that the fatigue

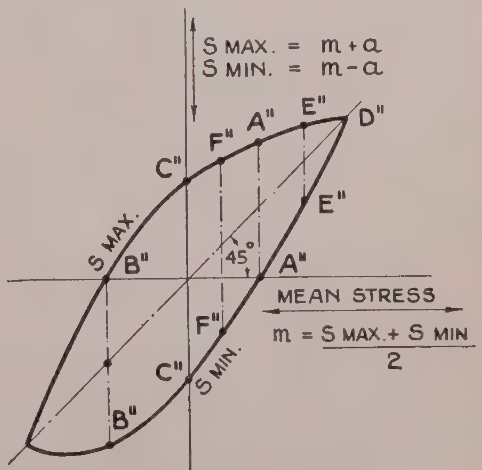

Fig. 3.—Schematic Smith diagram

strength of the material is enormously reduced. It may be disputed that there is actually an endurance limit for ferrous materials subjected to fatigue in dry air, but an endurance limit certainly does not exist even for ferrous materials if a corrosive medium has access to the metal. Fatigue loading emphasises the necessity for protection. Whereas a structure well protected against corrosion may have a very long if not an infinite life, the same structure if unprotected may fail much earlier for the same conditions of loading. Conversely, where a structure, statically loaded but exposed to the weather, may deteriorate gradually by general corrosion, failure from corrosion fatigue may take place under fatigue loading long before it might otherwise have become unserviceable from ordinary corrosion. Protection against corrosion therefore becomes particularly important for structures subject to fatigue. Periods of leaving the structure unprotected, even if they are relatively short in comparison with the total expected life, whilst it is subject to fatigue loading, may impair the fatigue strength permanently in consequence of the formation of fine corrosion fatigue cracks. This is the conclusion to be drawn from the work of Simnad at the Corrosion Department in Cambridge.[5] Hence it is not only important to protect structures subject to fatigue but to ensure that the protection is not

allowed to remain for any length of time in a deteriorated condition.

High Tensile Steel

The superiority of high tensile steel over mild steel may be partly or even entirely lost under fatigue loading for two reasons. The fatigue strength of high tensile steel decreases more rapidly with decrease in mean stress than that of mild steel, and it is generally more notch-sensitive than mild steel. In comparing two carbon steels of approximately 30 and 70 tons/in.^{2} tensile strength respectively, Roš[6] found : the yield points : 19 tons/in.^{2} and 27 tons/in.^{2} respectively ; the fatigue strength of rolled plate tested with the rolling surface not removed :

	M.S.	H.T.S.	
in tension:	16.8 tons/in.^{2}	22.2 tons/in.^{2}	from zero to maximum
in alternating	$\pm$10.0 tons/in.^{2}	$\pm$12 tons/in.^{2}	compression—tension.

Whereas the yield point in the high tensile steel is increased by 42 per cent. the tensile fatigue strength is increased by 32 per cent. and the alternating fatigue strength—zero mean stress is increased by only 20 (zero) per cent. From these figures it is already obvious that the permissible stresses for high tensile steels cannot be increased in proportion to the increase in yield point because the fatigue strength—particularly in the region of low mean stresses does not increase *pro rata*.

If stress concentrations are present the situation becomes even more unfavourable to high tensile steel. Stress concentrations in mild steel do not in general become fully effective. This means that the fatigue strength of mild steel is not reduced to, say, one-third, in the presence of a discontinuity with a stress concentration factor of 3, whereas in high tensile steel the stress concentration factor may become fully effective, as was shown by Phillips and Heywood in the paper already quoted. Under certain circumstances high tensile steel may not only be not superior to mild steel but may actually be inferior in its fatigue resistance.

This is particularly true if the high tensile steel is fabricated by welding. If high tensile steels are fabricated by welding without special precautions there is always a chance that a hard brittle structure is formed in the vicinity of the welds. The formations of such brittle constituents is frequently accompanied by the formation of small hardness cracks which will grow rapidly under fatigue loading, leading to complete destruction. If this happens, the fatigue resistance of the high tensile steel will be greatly inferior to that of mild steel. Even if precautions are taken to avoid the hardness cracks in welding, the fatigue resistance of high tensile steel when welded will be equal to that of welded mild steel under alternating loading.

RIVETED AND WELDED STRUCTURES

When one considers that a single hole may reduce the fatigue strength of a tension member by up to 60 per cent., it is astonishing that riveted structures have not on the whole seriously suffered from fatigue failures. The design principles for riveted structures have been evolved over a very long period of time and experience accumulated over decades—and it must be realised that decades may be necessary to gain experience on the effect of fatigue—has led to the development of empirical rules for design details—rivet pitch and grouping are only two of many—and to the adoption of permissible stresses which on the whole ensured reasonable safety against fatigue failure. Permissible stresses for riveted structures have gradually increased however during the past three decades and it is quite possible that their immunity from fatigue failure will come to an end.

The most highly stressed portions in every structure are the joints. We know now that friction is relied upon entirely for transmission of force through a riveted joint. This acts to some extent like a safety valve against too frequent overloading of the joint because the rivets will gradually work loose. In redundant structures, where a certain amount of stress redistribution is possible, rivets work loose under many applications of loading, and in this way the joint relieves itself of stress before fatigue failure takes place. Loose rivets in riveted structures must in fact be considered as a form of fatigue failure which prevents the development of fatigue cracks. It is also well-known from experience that where rivets have once worked loose they will often do so again after a time when replaced.

It must not be assumed however that fatigue failures in riveted structures have not occurred, though little information is publicly available. In a recent survey in America[7], 170 cases of fatigue failure in 83 spans of 50 riveted and 79 failures in 28 pin connected railway bridges were reported. Most of these failures occurred in floor beam hangers, some at axial stresses as low as 2.8 tons/in.^{2}. The fatigue failures were ascribed to certain deficiencies in detail design. This shows that fatigue resistance depends almost entirely on the proper design of details, also in riveted structures.

Another, more spectacular failure obviously caused by fatigue was discovered recently in the Manhattan suspension bridge*. The bridge, with a centre span of 1,470 ft., has four stiffening trusses of 24 ft. depth. The bridge carries a central roadway and two double-track subway lines on each side, with vehicular roadways arranged over the subway tracks.

The fatigue failure took place in the bottom chord of the easterly outboard truss at approximately the quarter point of the centre span nearest the Brooklyn pier. The chord is 3 ft. deep, 3 ft. wide, and is composed of two composite channel sections made up of two angles and two plates each, joined by flat double lacing. The maximum loads in the chords are 1,850 tons tension and 1,430 tons compression, giving rise to 13.5 tons/in.^{2} tension and 10.4 tons/in.^{2} compression, assuming a cross-section of 137 in.^{2}. (This figure, obtained from a drawing provided by Mr. Zurmuhlen, could not be checked.) The chord material is nickel steel, for which permissible stresses 40 per cent. greater than those used for mild steel have been advocated. Under fatigue loading, particularly when there is stress reversal, high tensile alloy steels show little superiority over mild steel and from this point of view these stress values must be considered extremely high for a bridge subject to fatigue loading.

The fracture, two views of which are shown in Figs. 4 and 5, occurred near the end of a chord splice where stress concentrations are bound to occur because of the sudden change in section, quite apart from the stress concentrations caused by rivet holes. The fracture passes through a number of rivet holes, but misses one or two.

In the opinion of the Consulting Engineers of the City of New York, the failure is due to unbalanced train loading on the truss which produces torsion in the chord. The bridge was opened in 1909. The fracture appeared after 43 years' service.

The reduction in strength under fatigue loading due to rivet holes is far larger than that corresponding to the

*The author is greatly indebted to Frederick H. Zurmuhlen, Commissioner of Public Works of the City of New York, for making this information available and for giving permission to publish it together with the photographs reproduced in Figs. 4 and 5.

mere reduction in cross-sectional area. Conversely, if rivet holes could be omitted it should be possible to save far more material than that which it is customary to add for holes. When it is considered how drastically the fatigue strength of a material is reduced by the presence of holes, and that, nevertheless, riveted structures designed on the basis of permissible stress roughly between 7 tons/in. and 9 tons/in. have not suffered unduly from fatigue failures, one is forced to the conclusion that if it were possible to eliminate rivet holes around rivet holes, welding is such a versatile method of construction that stress concentrations far more severe than those due to riveting can easily be introduced with the result that the welded structure may, for the same section sizes, be inferior in fatigue resistance to the riveted structure.

It must be remembered that a relatively small displacement of the *S-N* curve towards the *N*-axis may result in a large reduction in endurance. If curve A in Fig. 6 represents schematically the *S-N* diagram for a

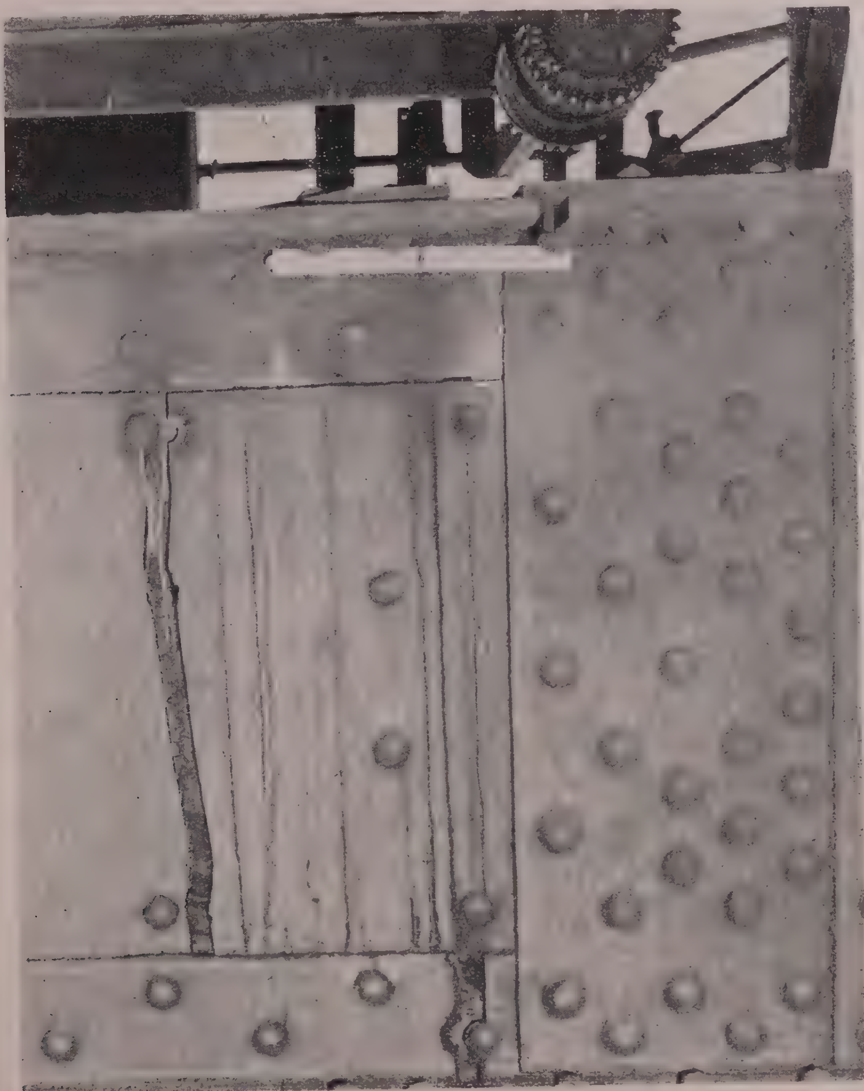

Fig. 4.—Fatigue fracture in bottom chord of Manhattan bridge

and the attendant stress concentrations altogether, the inherent fatigue strength of the material could be far more fully exploited. It should in fact be possible to design structures not containing rivet holes for at least twice the permissible stresses used for riveted structures, and this of course would mean a reduction of cross-sectional areas in tension by 50 per cent. The actual saving in structural weight, even if fatigue loading were the only design consideration, would of course be smaller because instability of compression members would have to be considered also and the permissible compressive stresses would have to be left unaltered.

Rivet holes are in fact eliminated when welding is used, and whilst it should be possible to effect appreciable reduction in the dimensions of members in consequence of the elimination of stress concentrations particular critical detail which has been designed for an average stress S_1 for which the life is virtually infinite, and if by some alteration in design the fatigue strength is reduced by a small amount, say 10 per cent., the resulting *S-N* curve *B* is displaced towards the *N* axis. The intercept with the line $S = S_1$ however occurs now at a point indicating only a fraction of the former endurance. Such comparatively small changes in the *S-N* curve can easily result from some change in detail, and such changes are more easily produced by welding than by riveting.

Where in riveted construction a particular connection can generally be made in one way only, welding frequently offers several alternatives which may differ greatly with regard to cost and also with regard to fatigue resistance. It is unfortunate that generally the

less expensive and straightforward types of joints and connections may show appreciable reductions in fatigue strength even when compared with riveting. This situation is responsible for the fact that far more research has been devoted to the problem of the fatigue strength of welded joints and connections than was ever devoted to similar investigations on riveting. It is true to say that scarcely anything was known about the fatigue strength of riveted joints until a great deal of information on the fatigue strength of welded joints had been obtained, and it became necessary to examine riveted joints also if only to obtain a datum for comparison.

The fact that so much work has been carried out on the fatigue strength of welded joints and is still being currently pursued, has created the impression that welding is suspect from the point of view of its endurance under fatigue loading. This is by no means the case. There is ample evidence to show that the fatigue strength of welded structures is at least equal and can be greatly superior to that of riveted structures, provided they are properly designed. This is an important proviso. All the research work going on is fundamentally devoted to this question of what constitutes proper design in welding from the point of view of fatigue. The problem is complex and the field to be covered is fairly extensive. There are different welding processes to be considered. Metallurgical effects due to the high temperatures reached in welding are of great importance ; there are different types of joints and the almost infinite number of ways in which the different types of joints can be modified, used singly or in combination.

There are also some fundamental differences between riveted and welded structures which make the latter more susceptible to serious damage by fatigue if improperly designed. In riveted structures the maximum stress concentrations occur round the rivet holes and these are removed from the edges where maximum fibre stresses occur. In welded structures the joints, and hence the stress concentrations, extend to the edges, and hence coincide with regions of maximum fibre stress.

Fig. 5.—Fatigue fracture in bottom chord of Manhattan bridge

It is far easier to introduce sharp corners in welded structures than in riveted structures. The joint shown in Fig. 7a and b between two plates of different width can be readily effected by a butt weld, with the result that a very high stress concentration is introduced at the corner. Such a joint if subjected to repetition of loading of quite modest magnitude, would fail from the corner in no time at all. (Fig. 7c.)

The same joint when made by riveting cannot be effected without allowing the plates to overlap or without using straps. Stress concentrations at the corner cannot arise because the free edge of the wider

plate must be stress-free. It must not be thought, however, that using welded straps instead of riveted straps will produce the same effect and eliminate the stress concentration at the corner. If side fillet welds are used as in Fig. 7d stress concentration will arise where the welds intersect with the edge of the wider

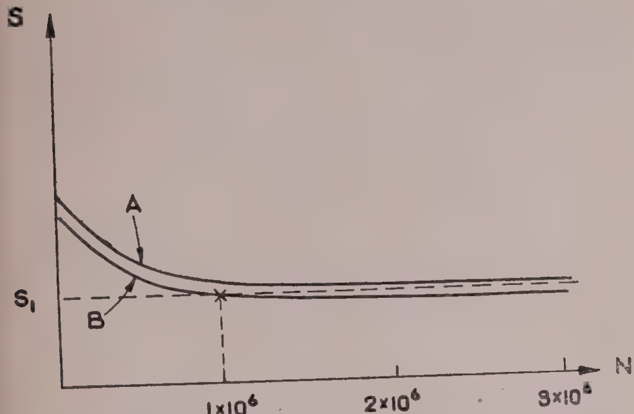

Fig. 6.—S-N diagram for slightly differing details. A small change in fatigue resistance may result in a large change in endurance

in a part of the plate with parallel sides of the same width as that of the narrower plate. This very simple example illustrates that :

(a) welding offers a greater number of solutions than riveting.

(b) what appear to be the straightforward and most economical solutions are unacceptable for fatigue conditions.

(c) solutions superior to riveting are obtainable only at the cost of more careful elaboration and possibly only at greater expense.

A further—from the point of view of fatigue undesirable—feature of the welded structure is its continuity. If a fatigue crack starts in one part of a riveted component, for instance in the flange of a plate girder, this may be completely severed without the crack continuing in either the flange angles or the web. The fatigue crack cannot jump a gap. In welded structures there are no gaps and the continuity of material offers a continuous pass once a crack has started. In a welded plate girder a fatigue crack might start going into the web before it has passed right through the flange. Interruption of continuity is desirable from the point of view of fatigue but difficult to achieve in welding without introducing at the same time geometric discontinuities.

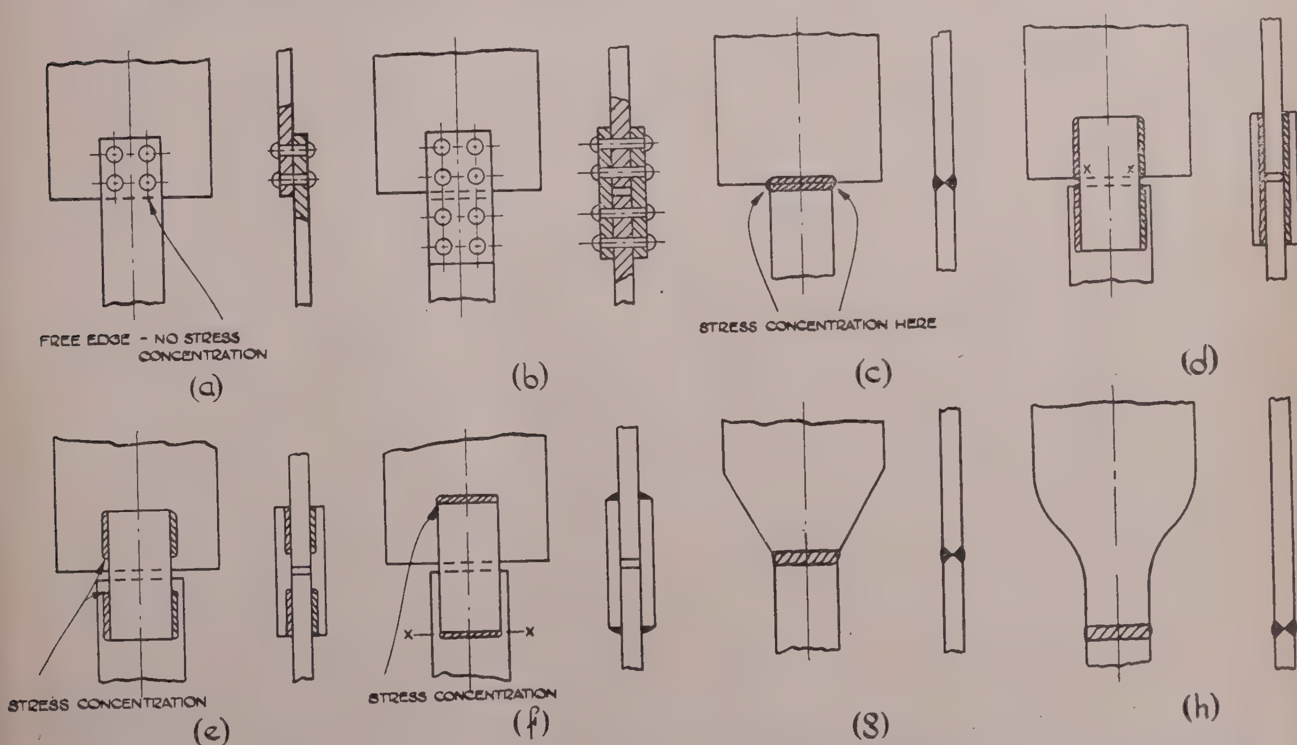

Fig. 7.—Riveted and welded alternatives of the same joint producing vastly different stress concentrations and hence exhibiting greatly differing fatigue strength

plate at the points marked x. If the side fillet welds are interrupted and stopped short of the plate edge as shown in Fig. 7c stress concentrations still occur at the ends of the welds. If side fillet welds are dispensed with altogether and end fillet welds are used as in Fig. 7f the stress concentrations will now occur at the ends of the straps either along the root or the toe of the fillet welds and these stress concentrations will be greater than those caused by rivet holes. The butt welded joint with the wider plate tapered as shown in 7g is preferable. Ideally, the wider plate should be shaped as shown in Fig. 7h and the butt weld should be placed

These considerations show why the subject of fatigue has assumed such importance with regard to welded structures subjected to many repetitions of loading.

FATIGUE STRENGTH OF WELDED JOINTS

The fatigue strength of a structural member depends on the stress concentrations introduced by the geometric discontinuities it contains. The joints between individual components making up a member and the joints between the members are in general the most severe discontinuities present. It follows that the joints assume overriding importance for the fatigue resistance

of the structure. In fact, it is true to say that the fatigue strength of a structure is determined by the design of its joints.

The structural engineer is accustomed to the design of joints capable of transmitting given loads. In this sense a joint can be weak or strong and it is considered good practice to make the joints just slightly stronger than the parts joined. This implies that the strength of the joint is under the designer's control, who can make it stronger if it is riveted by increasing the size of the rivets or their number, and if it is welded, by increasing the size of welds. This is perfectly true for conditions of static loading but entirely false for conditions of fatigue loading. The fatigue strength of joints cannot be increased by increasing the number of rivets or their size if the joint is riveted or by increasing the size of welds if the joint is welded. Strength of joints in the sense that it exists for conditions of static loading does not exist for fatigue. It is in fact not the strength of the joint one is concerned with but with the material joined whose fatigue strength is reduced by the geometric discontinuities of the joint. All types of joints produce stress concentrations, though some produce more severe stress concentration than others and hence produce greater reductions in the fatigue strength of the main material.

Butt Welded Joints

It might be thought that a butt weld, affording perfect continuity of the material, might enable the full fatigue strength of the main material to be realised. This is unfortunately not the case. Fatigue failure in members containing butt welds will take place at smaller stresses than those producing failure in the unjoined material. The fatigue failure will generally occur in the weld. In some cases it may develop along the edges of the butt weld and in some cases through the centre.

The fatigue strength of mild steel plate with the surface in the as rolled condition may vary in the region of between 14 tons/in.^{2} and 20 tons/in.^{2} for tension between zero and maximum (basic tensile fatigue strength) and between ± 7 to ± 14 tons/in.^{2} for stresses alternating between tensile and compression limits (alternating fatigue strength). Butt welds may reduce the basic tensile fatigue strength to values of between 7 tons/in.^{2} to 12 tons/in.^{2} and the alternating fatigue strength to between ± 4 tons/in.^{2} to ± 10 tons/in.^{2}. These ranges of fatigue strength deduced from the results of numerous investigations are disconcertingly large. The quality of the butt weld is of course of fundamental importance. In sound butt welds containing no internal defects the reduction in fatigue strength as compared to the plate is due to the notch effect of the reinforcement at the edges, and to surface ripples. If these discontinuities are removed by machining the reinforcement and the sealing run flush with the plate the fatigue strength of the plate can be approached very closely and the fatigue strength of such a butt weld will be greatly superior to that of a riveted joint.

Internal defects of which cracks and lack of fusion at the root are by far the most serious, may result in reductions of fatigue strength of up to 70 per cent., which is far more than corresponds to the mere reduction in cross-sectional area. Hempel[8] showed that the basic tensile fatigue strength of machined butt welds may be reduced from $12\frac{1}{2}$ tons/in.^{2} to 5 tons/in.^{2} by slag inclusions and to 4 tons/in.^{2} by defects in the root of the weld. According to Homès[9], 12 per cent. porosity may reduce the fatigue strength 50 per cent. from, say, $12\frac{1}{2}$ tons/in.^{2} to $6\frac{1}{4}$ tons/in.^{2}.

Provided the surfaces of butt welds are machined and their freedom from internal defects ensured, permissible stresses of 12 tons/in.^{2} could be readily accepted for tension and bending whereas 9 tons/in.^{2} in tension for riveted connections is certainly higher than is warranted by any fatigue test results for riveted joints obtained so far.

Fillet Welded Joints

The conventional method of designing fillet-welded joints is concerned only with the determination of the throat area for transmission of a given load. This is obtained on the basis of what is termed the permissible shear stress, which is generally at a fixed ratio to the permissible stress in tension. Depending on the specification, this procedure results in weld throat areas 40 to 100 per cent. greater than the area of the parts joined. The method's only merit is that it works and is very simple to use. Reasonably safe and economical joints result from its use but any relation to structural mechanics is purely coincidental.

Fillet welded joints in structures subject to fatigue loading can be designed on the same basis, but for the fatigue strength of the connection the choice of permissible shear stress, and hence the weld area, is immaterial within fairly wide limits. Whether the weld area provided is only just equal to the area of the parts joined, or twice or three times as large, is without much influence on the fatigue strength of the joint. It is erroneous to suppose that a fillet-welded joint can be strengthened by providing larger or longer welds. Provided there is a minimum throat area—a throat area equal to the cross-sectional area of the smaller part is ample—no increase in fatigue resistance results from increasing the weld size or the length of welds because the fatigue strength of the connection depends entirely on the stress concentrations introduced in the main material by the form of joint. Stress concentrations develop at the root and at the toe of fillet welds, and fatigue failure may start from either. Neither stress concentration can be alleviated appreciably by an increase in weld size and their effect can be diminished only by reducing the average stress in the material joined, that is, by increasing the plate or section size.

The basic tensile fatigue strength of rolled mild steel plate is reduced to between 4 tons/in.^{2} and 6.5 tons/in.^{2} and the alternating fatigue strength to between ± 2.5 and ± 4 tons/in.^{2} by fillet welds. It is clear that a member subject to many repetitions of tension stress between zero and a maximum equal to the permissible stress of 9 tons/in.^{2} could be expected to fail relatively early if it were joined by fillet welds. Its cross-sectional area would have to be doubled in order to reduce the average stress to a value of $4\frac{1}{2}$ tons/in.^{2} for which a life of approximately 2 million cycles could be expected.

This also shows that permissible stresses for members in structures subject to fatigue loading, and hence their dimensions, can really not be decided before a decision has been made how these members are to be fabricated and joined. Even if fillet welds are not transmitting any load but are merely used for attaching a stiffener, gusset, bracing member or bracket, a similar reduction in fatigue strength must be expected if the fillet welds run at right angles to the direction of stress, even if the part attached carries no stress at all. Hence, the practice of not welding stiffeners to the tension flange of a plate girder. In effect, it is better not to weld it to the compression flange either unless this is imperative to ensure stability. Fortunately, the fatigue strength of material subject to cyclic loading between zero and maximum compression is appreciably higher than the basic tensile fatigue strength, so that the reduction of

fatigue strength due to the presence of a fillet weld in a compression member is far less serious. Nevertheless, fatigue failures in the compression flanges of rolled steel joists have been obtained in fatigue tests carried out by Wilson[10] in America when the compression flanges were reinforced by cover plates attached by fillet welds.

The practice of using cover plates attached by fillet welds to reinforce butt welds of dubious quality is fairly widespread. Unless the butt welds are of very inferior quality, in which case they should in any case be condemned, such cover plates can only serve to reduce the fatigue strength to a value very little different from that which would have been obtained had fillet-welded cover straps without butt welds been used to effect the joint. Severe stress concentrations will arise at the ends of the cover straps as shown in Fig. 8, where the results of stress measurements along a beam thus " reinforced " and subject to bending stresses are plotted.

Some worthwhile improvement in the fatigue strength of fillet welds is obtained by using 30° fillets instead of

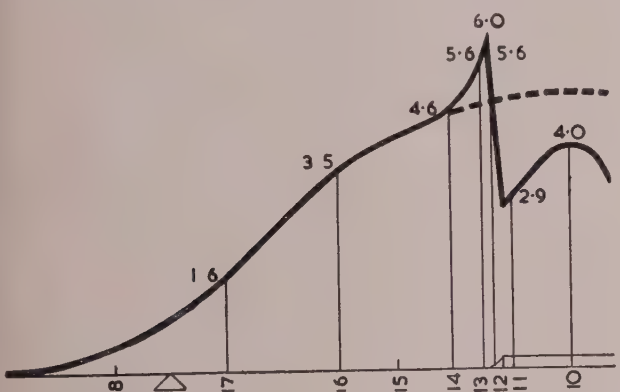

Fig. 8.—Measured stress concentration at end of cover plate of beam flange

45° fillets, and in important applications where there is no alternative to the use of fillet welds the shaping of 30° fillet welds to a concave contour so that all traces of undercut along the toe of the fillet weld are removed and the weld contour blends smoothly into the surface is recommended. Such joints will still not attain the fatigue strength of butt welds of good quality and fatigue failures of which one is shown in cross-section in Fig. 9 will occur at the toe of the fillet weld, albeit at higher stresses. The shaping can be readily and rapidly carried out with tungsten carbide files or burrs.

DESIGN CONSIDERATIONS

The only indirect reference to fatigue in any British structural specification is clause 19 of B.S. 153, 1937. This rule goes back so many years that few engineers nowadays understand it and some perhaps are not even awaré that it has anything to do with fatigue. Its effect is to provide a larger cross-sectional area for those members in which the stresses vary between tension and compression. The clause thus obviously takes account of the fact that the fatigue strength in tension for mean stresses smaller than half the maximum stress is smaller than the fatigue strength in tension for mean stresses greater than half the maximum. The clause is a very rough and ready rule which ignores two important facts but seems to have worked perfectly well for riveted bridges. It completely ignores that fatigue occurs also as a result of many stress applications entirely in the tensile region, that is without reversal, and more important that the fatigue resistance of members is determined largely by the details of their end connections. The only reference to end connections is contained in that part of the clause which requires them to be designed for a stress equal to the sum of the maximum tension and maximum compression. This implies that an increase in the number of size of rivets or both increases the fatigue strength which we now know to be untrue. This clause, despite its serious shortcomings, was satisfactory because it so happened that the permissible stresses specified for tensile loading did not greatly exceed the basic tensile fatigue strength of riveted connections so that fatigue failure in pure tension could in fact be ignored as long as the permissible stresses were not exceeded. Moreover, the maximum stress concentrations possible in riveted construction are probably those caused by rivet holes, so that stress concentrations due to the design of detail need not be considered.

Observance of the same clause for welded construction would provide no safeguard whatsoever because fatigue failure for repeated tension with welded connections

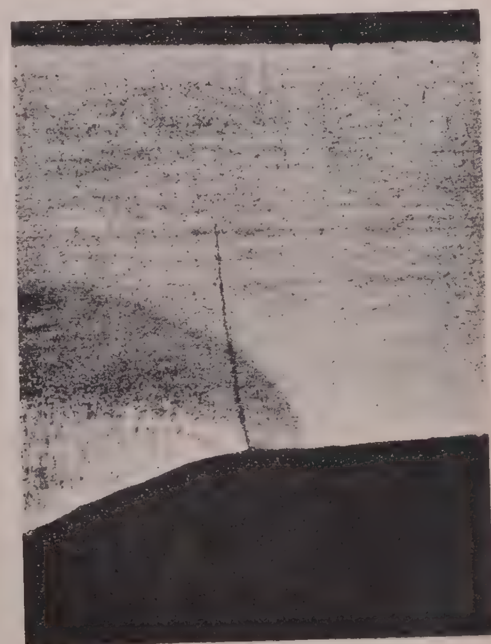

Fig. 9.—Fatigue failure at toe of machined, concave fillet weld. Note difference in metallurgical structure at point where crack starts

may occur in the purely tensile region at smaller than the permissible stresses, and even with butt welds the details may incorporate such severe stress concentrations that fatigue failure would occur if the permissible stress was applied two million times.

The situation in welded construction is in fact so complex that it is impossible to deal with it satisfactorily in a standard specification. Since there is a large difference between fillet-welded and butt-welded connections it is possible to take account of this by requiring the permissible stresses to be reduced by different amounts for butt-welded and for fillet-welded connections. Since the fatigue strength is a function of mean stress the reduction factor will depend on the ratio of maximum stress to minimum stress. The reduction factor will be a maximum for alternating stresses where the ratio $\frac{\text{maximum stress}}{\text{minimum stress}} = -1$ and will gradually diminish for increasing values of this ratio.

This procedure has been adopted in certain Continental specifications and will be incorporated in the new edition of B.S. 153.

It would be a mistake to assume that this is all that is necessary to ensure freedom from fatigue failure. The design of butt-welded or fillet-welded joints with the corresponding range of permissible stresses for the members still leaves plenty of scope to introduce details in design which may initiate fatigue failure. In specifications one has in general only the choice between outright prohibition or penalisation by reduced permissible stresses to ensure freedom from undesirable details and defects in design. Both methods are too crude to deal satisfactorily with the fatigue problem. Even when reduced permissible stresses are used, great care must be exercised in the design of detail. There is neither sufficient experience nor sufficient laboratory evidence to lay down comprehensive rules for the design of details even if such restrictions on the designer may to some appear desirable and necessary. Departure from what must be considered good practice in the light of our present-day knowledge does not, where fatigue is concerned, result in early failure and does therefore not immediately reflect on the designer's competence and reputation. His sins of commission or omission may not become evident until he has been dead for some time. In view of the uncertainties of our present-day knowledge the incentive to produce details of high fatigue resistance is therefore not very powerful, particularly as the design of such details may severely tax a designer's ingenuity if it is not to tax severely his clients' pockets. It must not be thought however that fatigue failures always take a long time to develop. Some appear surprisingly early, particularly if serious mistakes in design have been made.

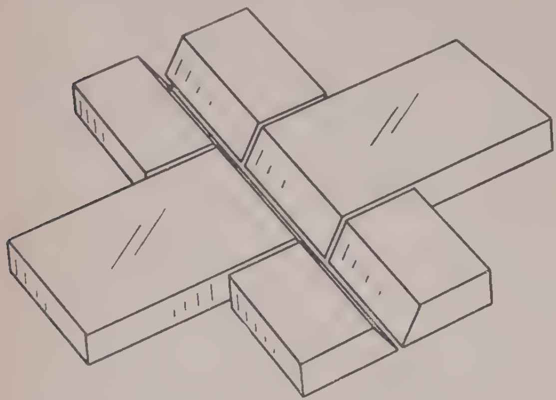

Fig. 10.—Extension pieces (run-off tabs) for butt welds containing beginning and end of weld. These should not be tack welded to the plate in which the butt weld is to be made and cut off after completion of weld

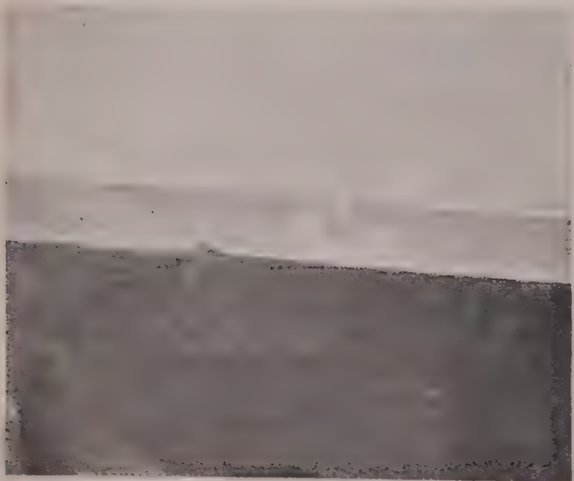

Fig. 11.—Notch left in edge of flange plate after removal of tack weld by machining. Such notches due to undercut in tack welding are favourite starting points of fatigue cracks

The most important requirement for a designer of welded structures subject to fatigue loading is to become intensely "notch conscious." The strict avoidance of all notch effects must be the first consideration. Secondary stresses, particularly those due to eccentricity of connections, may have the same result as stress concentration and should as far as possible be avoided. Stress arising in one member in consequence of the deformation of another member connected to it, so-called deformations stresses, should also be avoided. There are certain details to which particular attention must be paid. Though there are of course many common problems in plate structures and lattice structures it is best to consider these two main groups of structures separately.

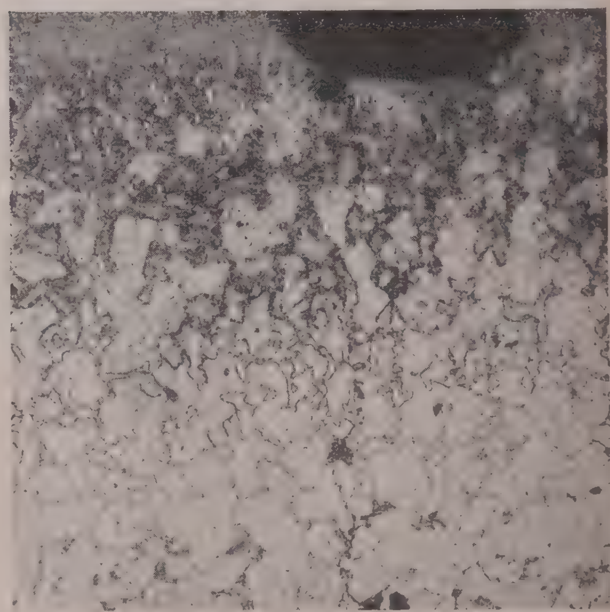

Fig. 12.—Enlarged cross section through stray flash in mild steel, showing severely hardened structure and fatigue crack starting from hardness crack

Plate Structures

Flange plates should not be built up. It is preferable when economic considerations warrant this to use flange plates of variable thickness or variable width joined by full penetration butt welds. These should be placed into the parallel portion and the thicker or wider plate tapered to the thickness or width of the thinner or narrower plate.

Butt welds in flange plates should under no circumstances be provided with cover plates over the width or the edges. Because crater cracks may appear at the end of a butt weld the beginning and the end of a butt weld should be contained in temporary extension pieces which are cut off when the weld is completed. These extension pieces or run-off tabs illustrated in Fig. 10, should under no circumstances be welded or tacked to

the flanges. Broken tack welds and the notches they leave in the surface of the plates even after machining, as shown in Fig. 11, are starting points for fatigue cracks and may produce enormous reduction in fatigue life quite out of proportion to their size. Altogether, tack welds which are not welded over afterwards should be avoided.

Stray flashes, such as are produced either accidentally or when the arc is started, should be most carefully avoided outside the welds themselves. In consequence of the very high temperature reached and the very small heat input in such an arc spot even a very soft steel will harden severely, and the hardening will

Fig. 13.—Undesirable dovetailing ("egg box" construction). The sharp notch may result in premature fatigue failure starting from the corners

generally be accompanied by fine hardness cracks, as can be seen in Fig. 12, which is a magnified cross-section of such a stray flash. These cracks will grow with every loading cycle, and as a result the fatigue strength will be much reduced.

Dovetailing of members, the so-called "egg-box" construction, as illustrated in Fig. 13, should be carefully avoided. At the bottom of the "slot" a very severe internal notch is left from the corner of which fatigue failure may readily start at comparatively small stresses. The slotting of one member to pass another member

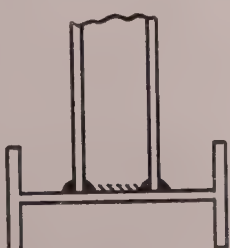

Fig. 14.—Welding a member to a flexible plate without stiffening the plate at the point of attachment is undesirable because of the risk of fatigue failure in the plate

through leaves the same defect, even if the member passing through the slot is welded on both sides by continuous fillet welds to the member which it transfixes.

No member should ever be welded directly to the face of a thin plate without preventing the deformation of the plate by a stiffener. The cross girder welded to the web of the main girder, as shown in Fig. 14, is restrained at the ends, and when the cross girder deflects it will distort the web. Unless this is prevented by a stiffener fatigue cracks will occur in the web immediately above and below the main girder. The joint between main girder and cross girders in welded railway bridges is a design problem which so far has not found a satisfactory solution. Under no circumstances should the bottom flange of the cross girder be attached to the flange of the main girder. In fact, the tension flange of a plate girder should be completely free of all welded-on attachments. Particularly reprehensible is the butt welding of two flanges at right-angles, as shown in Fig. 15a, where two points of great danger are created in the re-entrant angles. This dangerous situation cannot be alleviated by welding in circular gussets, as shown in Fig. 15b. The points of danger are merely moved to the ends of the welds attaching the gussets. If such joints cannot be avoided the only satisfactory solution is a separately fabricated transition piece to which the three members can be attached separately by butt welds.

A number of fatigue tests were recently carried out by the author with four different types of corners for

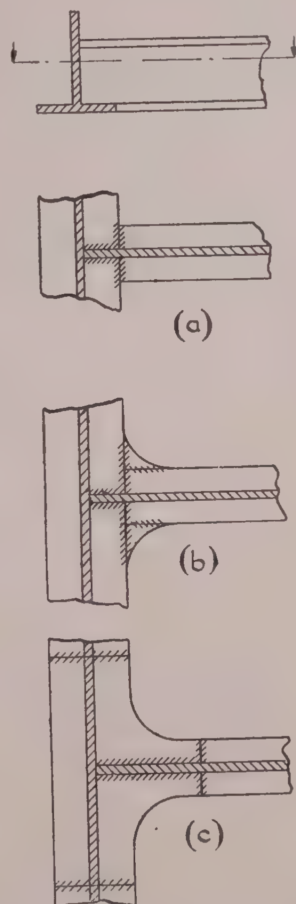

Fig. 15.—Butt joint between girders. The sharp corners of joint "a" are very dangerous. Insertion of circular gussets "b" is no improvement. Detail "c" is ideal from point of view of fatigue

otherwise identical rigid box section frames. Two of the test results are of interest. The sharp corner shown in Fig. 16 subject to bending, opening the corner, withstood 2 million cycles of a certain load "x," a further 2 million cycles at a load of $1.5x$, and failed after a further 75,000 cycles at a load $1\frac{3}{4}x$. The round corner shown in Fig. 17 withstood 2 million cycles at x, plus a further 2 million at $1.5x$, a further 2 million at $1\frac{3}{4}x$, a further 2 million at $2x$, and finally broke after a further 5 million cycles at $2\frac{1}{4}x$, which was the maximum load the testing machine could produce. The total life of the round corner when subject to loads above $1\frac{3}{4}x$ was over 7 million cycles, whereas the sharp corner broke after only 75,000 cycles at $1\frac{3}{4}x$.

These experiments show the great superiority of curved corners over sharp corners under fatigue loading conditions. It is of great importance, however, that

the joint between flanges and webs should be a full penetration continuous butt weld of good quality. Otherwise fatigue failure will occur in consequence of the radial stresses and the transverse bending stresses of the flange, either by failure of the weld between web

Fig. 16.—Sharp corner in box section frame subject to fatigue tests opening the corners

Life at load	x	2 million cycles
	$1.5x$	2 million cycles
	$1.75x$	75,000 cycles
		total life 4,075,000 cycles
		maximum load $1.75x$

and flange or by the development of a fatigue crack in the flange running along the web round the corner.

Flanges and webs of plate girders or box girders should preferably be joined by butt welds. Fillet welds are permissible but intermittent fillet welds should not be permitted. At the ends of intermittent fillet welds very high stress concentration will occur which may produce fatigue cracks. This also applies to the welding of crane rails to gantries. If a crane rail is welded to the flange it will act as part of the section. Fatigue cracks may start at the ends of the intermittent fillet welds, preferably in the rail, though there is always the possibility of cracks travelling into the flanges as well.

It has been believed at one time that various special flange profiles popular on the Continent offer some advantage with regard to their fatigue resistance. Bierett[11] carried out fatigue tests with six different types of plate girders incorporating four different types of flanges. He found no significant difference in their fatigue strength.

Truss and Lattice Structures

The low fatigue strength of members joined by fillet welds creates particular difficulties in the economic design of trusses and lattice structures. The simplest joint between members is undoubtedly obtained by fillet welding one member directly to another without gusset plates. If such structures are designed on the basis of permissible stresses at present in use it would be optimistic to expect freedom from fatigue failure for very long if they are really subjected to their maximum design stresses at great frequency.

This conclusion, to which one is forced on the basis of laboratory experiments, seems contradicted by the success of the Bailey bridge, which is perhaps the outstanding example of a welded truss bridge, the more remarkable for the fact that the stresses used in its design are appreciably higher than those which would find general acceptance amongst bridge designers even for the type of high tensile steel used. It must not be forgotten, however, that the Bailey bridge is essentially a road bridge for which it is reasonable to ignore fatigue altogether and that a period of one decade is perhaps not long enough to arrive at final assessments with regard to fatigue.

There are certain indications—though the evidence is incomplete and not fully conclusive—that under certain circumstances the fatigue resistance of fillet welded joints may be equal to that of butt welds. Campus[12] carried out fatigue tests with fillet-welded joints which seemed to indicate that the fatigue resistance of fillet-welded joints decreased less rapidly with increasing mean stress than that of butt welds. This is schematically shown in Fig. 18. For zero mean stress and for zero minimum stress fillet welds are undoubtedly much inferior to butt welds, but as the minimum stress increases the fatigue strength of butt welds decreases more rapidly than that of fillet welds. There may be a region of high mean stress where the fatigue resistance of fillet welds is equal to that of butt welds. This is perhaps not so surprising since with increasing mean stress one approaches more and more closely to conditions of static loading and under static loading fillet welds are of course as strong as butt welds and can be made appreciably stronger. Further research is needed

Fig. 17.—Round corner in box section frame subject to fatigue test opening the corner

Life at load	x	2 million cycles
	$1.5x$	2 ,, ,,
	$1.75x$	2 ,, ,,
	$2x$	2 ,, ,,
	$2.25x$	5 ,, ,,
		total life 13 million cycles
		maximum load $2.25x$

to establish with certainty that this is in fact the case. If it were so it would offer the possibility of using fillet welds at any rate for those members in trusses where the dead load stress is equal or greater than half the live load stress.

The use of butt welds in place of fillet welds by itself is, however, insufficient if sharp corners and re-entrant angles similar to those customary in riveted construction are permitted to occur, as is shown in Fig. 19a. This

requirement taxes the ingenuity of the designer severely because it is no use to fake gradual transitions by welding curved gussets in between the members. The best solution is the use of separate fabricated transition pieces at each node to which the members can be attached readily by butt welding.

Roš[13] carried out fatigue tests with welded trusses constructed in this way. The types of joints used for

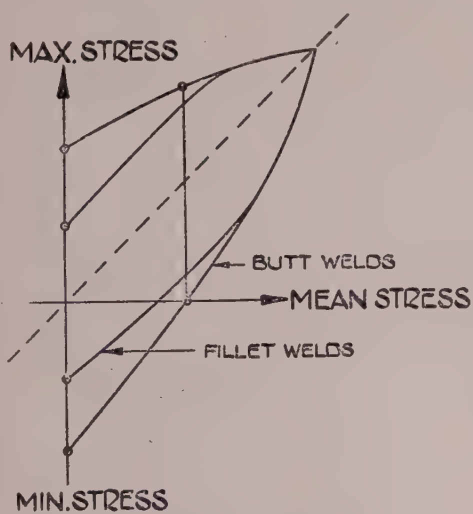

Fig. 18.—Hypothetical Smith diagram for fillet welds and butt welds

the two types of trusses are shown in Figs. 20 and 21. Similar riveted trusses were tested for comparison and it was found that the fatigue strength of the welded trusses was fully equal to that of riveted trusses. Welded trusses fabricated in this way may not be as economical as riveted trusses. If the design could be arranged in such a way that there are a large number of identical junctions it may be economical to use steel castings

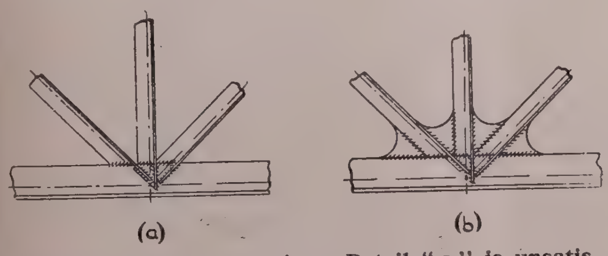

Fig. 19.—Truss connections, Detail "a" is unsatisfactory for fatigue conditions because of sharp corners and the use of butt welds in itself is no improvement. The welding in of gussets "b" does not improve the fatigue strength

for the transition pieces. No difficulty of welding the members to such castings need arise provided the carbon content does not exceed 0.2 per cent. and other elements such as sulphur phosphorus, manganese and silicon do not exceed those values usual in rolled sections.

The all-welded truss or lattice girder if subjected to fatigue loading may not be practical at present from the point of view of cost, particularly if it is large. The use of fabricated members joined by riveting or bolting with high tensile steel bolts is practicable. In this case it is comparatively easy without introducing serious stress concentrations to provide such members with enlarged or thicker ends to compensate for rivet hole deductions, so that the additional material need not be provided for the full length of the members.

The difficulties in the design of welded structures subject to fatigue loading are a consequence of the versatility of the process. They present a challenge to the ingenuity of the designer, the fabricator and the man in the laboratory to use this versatility in overcoming them. They cannot be overcome by ignoring them. Nor can progress be made and experience be gained if for lack of courage and enterprise we refrain

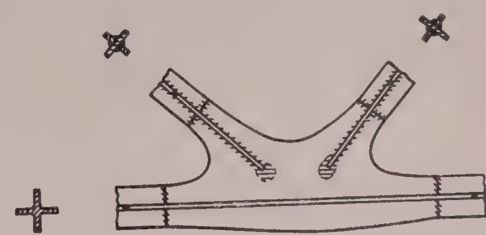

Fig. 20.—Typical joint in trusses tested by Ros

from using a most advantageous method of construction because there are certain difficulties and complications when fatigue failure cannot be safely ignored.

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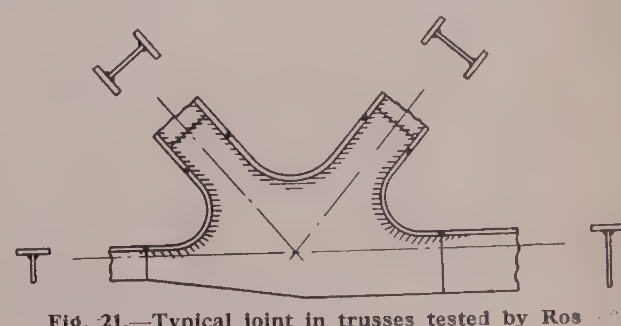

Fig. 21.—Typical joint in trusses tested by Ros

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Soil Mechanics in Relation to Structural Engineering*

Written Discussion on Mr. P. L. Capper's Paper

Commenting on Mr. S. P. Banerjee's contribution to the discussion, Professor A. L. L. BAKER writes : On page 24, reference 21, it is stated that moments M_b and $M_b + M_c$ are *approximately* proportional to their corresponding deflections. The closeness of the approximation depends on the degree of similarity of shape of the bending moment diagrams. When the ratio of $\frac{Y_b}{M_b}$ is greater than $\frac{Y_c}{M_c}$ which is usual on account of the relative shapes of the M_b and M_c diagrams, the approximation is on the safe side. Even in special cases, when this is not true, the vagueness of K values for the soil and their distribution, and EI values for beams and their distribution necessitates the use of extreme safe limiting values, which justifies using the above convenient approximation.

Mr. Banerjee refers to an isolated footing example which is a special case, and therefore does not invalidate the general principle. Even in this case, if the pressure distribution is assumed to be slightly more concentrated under the column than when a trapezoidal form is assumed, which is reasonable, there is good agreement between the alternative methods of calculation.

Mr. P. L. CAPPER writes : The author wishes to thank Mr. Banerjee for his written comments. He would like to point out that in the paper he referred to Professor A. L. L. Baker's "Soil Line Method of Design of Raft Foundations" as an example of the balance of the deformation characteristics of the soil and raft. The detailed calculation of the pressures and moments induced was outside the scope of the paper. With reference to the Modulus of Subgrade Reaction, the author does not consider this to be the ideal coefficient to define the pressure deformation characteristics of soil, since it is based on an arbitrary range of deformation and on an arbitrary size of bearing plate.

**Read before the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1, on Thursday, February 26th, 1953, Mr. E. Granter, B.Sc.(Eng.), M.I.C.E., M.I.Struct.E. (President), in the Chair. Published in* THE STRUCTURAL ENGINEER, *Vol. XXXI, No. 2. pp. 47-61 (Feb. 1953). See also Discussion published in* THE STRUCTURAL ENGINEER, *Vol. XXXI, No. 7. pp. 190-6 (July, 1953), and No. 9, p. 262 (September, 1953).*

Book Reviews

Some Examples of Reinforced Concrete Design, 4th Edition, by Oscar Faber. (Oxford University Press, 1952.) 90 + x pp., 8½ in. × 5½ in. 9s. 6d.

Written as a companion volume to "Reinforced Concrete Simply Explained," by the same author, this book contains examples of reinforced concrete design ranging from a complete framed building to retaining walls and water-towers.

The calculations are clearly explained and the text is supplemented with numerous illustrations. Of particular value to the student are the large-scale diagrams showing the arrangement of reinforcing bars at the intersection of beams and columns.

Compressive Stresses in the webs of continuous tee-beams, which receive scant attention in many text-books on this subject, are dealt with in some detail. The design of systems of shear reinforcement also is given prominence, though in a book of simple examples more use might have been made of stirrups as an alternative to the somewhat complicated arrangements of inclined bars.

One criticism that the writer would make is that the book would have been more readable had it been made more complete in itself without the need for such frequent reference to the previous volume. C. H. H.

The Design of Dams, by A. Bourgin. (Translated from the French by F. F. Fergusson.) (London : Pitman, 1953.) 344 + xiii pp., 8½ in. × 5½ in. 45s.

The author is a distinguished French engineer and Professor in the school for Hydraulic Engineers in Grenoble. The book is a distinct acquisition to all those concerned with the design of dams, especially for structures over, say, 100 ft. in height, and gives a clear exposition of the fundamentals involved ; there has been so much literature on arch and gravity arch dams, that a concise sorting-out and summing up of present design methods is very welcome. Two omissions under the Bibliography of Arch Dams, however, are somewhat noticeable—the notable contributions of the Italians and the U.S.A. engineers to the analysis of this complex subject.

The author has divided his work under four headings : (1) theoretical, (2) gravity dams, (3) arch dams, and (4) counterfort and hollow dams. His starting point is a wise one, since the subject-matter is highly mathematical, based on the theory of elasticity ; without knowledge of the basic principles the reading matter would be difficult to follow. He has, therefore, made the task easier by a general outline of the fundamental analysis, with its application to beams and arches.

His treatment of gravity dams deals efficiently and clearly with modern methods of analysis, and includes the effect of earthquakes. He regards the arch dam by inference as the safest type of construction under these shocks, but its overall superiority is a matter of opinion in spite of the reduced inertia effect. Its greater resistance to sliding or overturning compared with a gravity dam is undeniable.

The method of computing the anchoring of existing dams by post-stressing, as carried out by M. Coyne, is clearly given. This technique may well have even greater applications in the future.

A short chapter on deformation of gravity dams helps to fill a gap, since this is of vital importance in the case of high dams.

The treatment of arch dams is of particular interest, with its great complexity. The work covers arches of constant angle and radius, and variable radius ; a circular encastre arch of constant thickness, except for fillets at the extremities, is fully studied, including the buckling factor or coefficient of slenderness.

Hollow and counterfort dams have a chapter devoted to them, but their lighter section renders them possibly more vulnerable in war ; their economic and structural advantages are otherwise overwhelming where the site is suitable.

The work is a real contribution to the literature on dams, and the translator is to be congratulated. Criticism of using words such as " homethety " and " encastre " (long well accepted) when simple words such as " similarity " and " fixed ends " are available is perhaps unfair, but the first word baffled the reviewer, and was only solved by a French Technical Dictionary. The book is one which every engineer engaged on dam design would find of great value for references. J. G. B.

Modern Practical Masonry, by E. G. Warland (2nd Edition, London : Pitman, 1953 ; 270 + xviii pp. ; 10 in. × 7½ in. 50s.).

The opening chapter of this book on Details and Construction acquaints the reader with the requirements of modern building work and how it is associated with masonry. Beginning with Foundations and Footings—always the most important—copious illustrations and sketches show how masonry is associated with the construction of concrete rafts, piers and stanchions. Walls of masonry with their joints and finishes are then explained together with work on lintols and windows including metal connections. Soffits, gables and buttresses with all kinds of arches are illustrated and detailed very well and are arranged in such a way as to be a pleasurable help to the specialist and instructive preparation for the keen student of the building industry.

Section I of this book is concluded with nearly sixty photographs of actual work in masonry construction dealing with the materials used : stones, their cutting and handling, also limes and cements.

Section II, which deals with an important part of the work, describes the geometry of masonry and its setting-out and gives both plane and solid geometry in its practical aspects as required by the masonry draughtsman. This branch of the mason's art has an absorbing interest for the student of masonry and building construction who has a liking for geometrical problems. Readers who are desirous of becoming experts in geometrical masonry will enjoy this concentrated study and its many practical examples with the setting-out drawings which are included as insets. The variety of the practical examples in this section of the book entirely removes the tediousness which is usually associated with geometry. H. P. S.

Fleming Bros. (Structural Engineers) Ltd., Pocket Section Book, 8th Edition. (Glasgow and London, 1952.) 646 pp. and photos. 10s. 6d. 4½ in. × 2¾ in.

In the 8th Edition of this useful Steel Section book, which really is " pocket-size," all tables have been revised to conform to the new B.S.S. 449, and at the same time all other data has been revised in accordance with up-to-date practice. In addition, the section on welding has been enlarged by the addition of coefficient tables of bending moments of double-span rigid frames.

Handbook for Constructional Engineers. (Middlesbrough, England : Dorman London & Co., Ltd., 1952.) 7½ in. × 5 in. Limited issue.

This useful handbook contains tables relating to steel and information regarding the products and manufactures of Dorman Long & Co. The new edition is considerably enlarged by the introduction of new information and tables, and has been re-arranged into fifteen sections for quick reference, each section being well-indexed.

Tables of safe loads for beams, compound stanchions and struts, prepared under the joint auspices of the British Constructional Steelwork Association and the British Steel Makers are reproduced, and extracts from British Standard Specifications and from the Ministry of Transport (Roads Department) Regulations are given.

In addition, there is a section giving weights and measures, mensuration and trigonometry, logarithmic and other tables.

Principles and Practice of Prestressed Concrete, Vol. I, by P. W. Abeles, 2nd Edition, Revised. (London : Crosby, Lockwood, 1952) 116 pp., 9¾ in. × 7¼ in., 85 Figs., 11 tables. 21s.

This is the second edition of a book given a very favourable reception in 1949, which deals with the principles, analysis, design and application of prestressed concrete, and details of various methods of research, mainly with reference to prestressed concrete beams.

The descriptive part of the book is mainly as in the first edition since a second volume, dealing with new developments, is in course of preparation and will be styled " Volume Two." The revisions of importance are in detail ; thus the losses due to shrinkage and creep are interpreted on the basis of the " First Report on Prestressed Concrete," prepared by an *ad-hoc* Committee of the Institution of Structural Engineers, and on the German draft regulations on Prestressed Concrete, the permissible stresses adjusted and the failure conditions described in more detail, and the principal stresses are discussed on a more general basis. In addition, the notation has been revised to agree with the recommendations of the above-mentioned Report.

Extracts from the " First Report on Prestressed Concrete," and from the " German Draft Regulations, 1950," are given in the Appendix.

Underpinning and Strengthening of Structures, by L. E. Hunter, M.Sc., M.I.Struct.E., A.M.I.C.E. (London : Contractors Record, Ltd., 1952), 162 pp., 4½ in. × 6 in., 18 plates ; 110 figs., price 25s. net.

This book constitutes a very useful guide to methods of underpinning and strengthening structures of many types and its value is increased by chapters on " Foundations in subsoil subjected to heat " and " Settlement and subsidence." The many design examples given show clearly that the author has based his treatise on wide practical experience and generally they are not difficult to follow although, as is mentioned in the preface, space limitations precluded full details of all the calculations. The text is lavishly illustrated throughout by many line figures and photographs.

The writing contains many ambiguities which can usually be resolved but which detract from the value of the book, particularly to readers with little experience of the subjects covered. This is especially true of the final chapter ; for example, one might conclude from the paragraph beginning at the bottom of page 155 that it is useless to drive precast piles through a stratum liable to consolidation settlement to a firm stratum below.

Both designing engineers and contractors will find this book to be a valuable practical work of reference. For rapid use as such a minor but definite improvement would be to print the chapter titles instead of the book title on the top of all the right-hand pages in future additions. F. N. B.

Institution Notices and Proceedings

ORDINARY GENERAL MEETING

An Ordinary General Meeting of the Institution of Structural Engineers was held at 11, Upper Belgrave Street, London, S.W.1, on Thursday, February 25th, 1954, at 5.55 p.m., Lt.-Colonel R. F. Galbraith, M.C., B.Sc., M.I.Struct.E., A.M.I.C.E. (President), in the Chair.

The following members were elected in accordance with the Bye-Laws. Will members kindly note that the elections as tabulated below should be referred to when consulting the Year Book for evidence of Membership.

STUDENTS

BINGHAM, David Malcolm, of Liverpool.
BLUFF, Roy, of Manchester.
BRETT, Peter Ronald, of Reigate, Surrey.
CHALK, Edward Charles Patrick, of Kingston-upon-Thames, Surrey.
CHRISTIE, James, of Glasgow.
DANBY, Peter, of Ripon, Yorks.
FOZZARD, John, of London.
GRIFFIN, Ernest John, of Hounslow West, Middlesex.
HUNT, Dennis William, of Sutton, Surrey.
ROSS, Allan Reginald, of Manchester.
RUTHVEN, Brian, of Bradford, Yorkshire.
SHERBOURNE, Archibald Norbert, of North Wembley, Middlesex.
TAN YORK HING, of Johore, Malaya.
TAYLOR, Harry, of Middleton, nr. Manchester.
THORBURN, Thomas Hunter, of Cleland, Lanarkshire.
TYNAN, Derek Francis, of Kenton, Middlesex.
VAUGHAN, Reginald Ritchie Norman, of Nairobi, Kenya.
WHEATON, Roger Cecil, of Ilford, Essex.

GRADUATES

ARCEIVALA, Solie Jal, B.E.(Civil) Bombay, of Bombay, India.
BANERJI, Biswa Nath, B.E.(Civil) Calcutta, of London.
BONNETT, Clifford Frederick, of East Barnet. Herts.
CUTHBERT, Keith Henry Robson, B.Sc.(Civil) Leeds, of Northallerton, Yorks.
FRASER, James Douglas, of Liverpool.
JUDD, Henry Courtenay, of Cardiff, Glamorgan.
KULKARNI, Gopal Sadashiv, B.E.(Civil) Bombay, of Dist. Nasik, Bombay State, India.
LEE, Terence Godfrey, of Mitcham, Surrey.
MERRAN, Sidney, of London.
SHIPPEN, Clive, of London.
SMITH, John Frank McLaren, of Harrow, Middlesex.
STYLES, Ivor, of Mickleover, nr. Derby.
TATTERSALL, Gilbert, of Coventry, Warwickshire.
TAYLOR, Brian, of Manchester.
WOOD, Raymond John, of Gillingham, Kent.

ASSOCIATE-MEMBER

BALSARA, Rustom Jehangir, of Bombay, India.

TRANSFERS

Student to Graduate

DUERDEN, Thomas Brian, of Chigwell, Essex.

Graduates to Associate-Members

BARTAK, Andrzej Jozef Jerzy, of Harlow, Essex.
JENKINS, Robert William, of Cheam, Surrey.
MORROW, James Robert, B.E.(Civil) New Zealand, of Porthcawl, Glamorgan.
SALTER, Terence Herbert, of Beckenham, Kent.

Associate-Members to Members

COLLINS, Wilfred Samuel Addicott, of London.
EALES, Frederick Charles, of Orpington, Kent.
GARDINER, Peter, of Nailsea, nr. Bristol.
HUNTLEY, Harry David, of Stanmore, Middlesex.

Members to Retired Members

BATEMAN, Brigadier Harold Henry, D.S.O., M.C., of Ightham. Kent.
DAIN, Bertram, of Bromley, Kent.
FARMER, Frank Quentery, F.R.I.B.A., of Amersham, Bucks.
GARDNER, George Anthony, O.B.E., of Chiddingfold, Surrey.
HARMES, Edward Andrew, of Hove, Sussex.
MORRIS, David John, M.I.C.E., of Swansea.
THOMAS, Frederick Arthur, M.I.Mech.E., of Jersey, C.I.

OBITUARY

The Council regret to announce the death of GEORGE HENRY GOODMAN, HUBERT HAUGHTON (Members), EDWARD ROE BROWN (Associate), JOHN VICTOR TRELAWNEY HENWOOD, ERNEST WILLIAM TURNER (Associate-Members).

RESIGNATIONS

Notification was given that the Council had accepted with regret the resignations of William HUNTER, Eli TAYLOR (Members) ; Herbert Hollings JACQUES, John BREBNER (Retired Members) ; William Eugene BLACKMORE, William Ernest PARKER, John SHAW (Associate-Members) ; Ronald William ALLPORT, James BOLLAND, John MARTYN, Victor Thomas POULTON, Keith Nelson WAIDE, Philip Michael WORTHINGTON (Graduates) ; Douglas Oliver MALTHOUSE, Colin Reginald REES, Lester William STOCK (Students).

EXAMINATIONS, JULY, 1954

The Examinations of the Institution will next be held at Centres in the United Kingdom and Overseas on July 13th and 14th, 1954 (Graduateship), and 15th and 16th (Associate-Membership).

FORTHCOMING MEETINGS

The following meetings will be held at 11, Upper Belgrave Street, London, S.W.1.

Thursday, April 22nd, 1954

Ordinary General Meeting for the election of members, 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when Dr. R. Weck, A.M.I.C.E., A.M.I.Mech.E., will give a paper on "Fatigue of Welded Structures."

Tuesday, May 18th, to Friday, May 21st, 1954

Summer Meeting at Birmingham.

Thursday, May 27th, 1954

Ordinary General Meeting for the election of members, followed by the Annual General Meeting of the Institution.

Annual General Meeting of the voting contributors to the Institution's Benevolent Fund.

Thursday, June 24th, 1954

Ordinary General Meeting for the election of members.

Members wishing to bring guests to the Ordinary Meeting announced above are requested to apply to the Secretary for tickets of admission.

INSTITUTION AWARDS

The following awards have been made for papers read before the Institution and at the Branches during the Session 1952-53 :—

LONDON (HEADQUARTERS) PRIZE

Mr. Donovan H. Lee, for a paper on "Prestressed Concrete Bridges and other Structures."

LANCASHIRE AND CHESHIRE BRANCH PRIZE

Dr. P. W. Rowe, for a paper on "Development in the Design of Sheet Pile Walls."

MIDLAND COUNTIES BRANCH PRIZE

Mr. S. M. Cooper, for a paper on "The Tacoma Narrows Bridge—Design Features and Collapse Investigations."

NORTHERN COUNTIES BRANCH PRIZE

Dr. D. M. Brotton, for a paper on "The Application of Relaxation Methods to the Solution of Problems in Structural Engineering."

NORTHERN IRELAND BRANCH PRIZE

Mr. J. C. Malcomson, for a paper on "Recent Developments in Prefabricated Concrete Structures."

WALES AND MONMOUTHSHIRE BRANCH

Colonel R. D. Heseltine, for a paper on "Post War Construction."

WESTERN COUNTIES BRANCH PRIZE

Mr. F. G. Clarke, for a paper on "Some Local Contracts and Welded Steelwork for a Bus Garage."

YORKSHIRE BRANCH PRIZE

Dr. S. Mackey, for a paper on "Secondary Stresses in Steel Bridge Girders."

REVISION OF BYE-LAWS

On January 12th, 1954, the Lords of Her Majesty's Most Honourable Privy Council approved the revisions to the Institution's Bye-Laws which had been adopted by the corporate members in Special General Meeting on October 9th, 1953.

Copies of the Charter and Bye-Laws may be obtained from the Secretary, 11, Upper Belgrave Street, London, S.W.1, price 2s.

RHODESIAN INSTITUTION OF ENGINEERS

The Council have received a copy of the Rhodesian Institution of Engineers (Private) Act, 1953, passed by the Legislature of the Colony of Southern Rhodesia, which provides for the establishment and incorporation of the Rhodesian Institution, and defines its powers and objects.

The Act recognises, *inter alia*, full Membership and Associate-Membership of the Institution of Structural Engineers as a qualification for full Membership and Associate-Membership respectively of the Rhodesian Institution.

Mr. K. G. Stevens, the Institution's Representative in Southern Rhodesia, has been appointed a member of the Inaugural Board set up under the terms of the Act.

BURSARIES IN CONCRETE TECHNOLOGY

Notice has been given by Imperial College of Science and Technology, Department of Civil Engineering, that the election to Bursaries in Concrete Technology tenable as from October, 1954, will take place in June, 1954.

Candidates must hold a degree in Engineering at the time of taking up the award, and must also have a good knowledge of the theory of structures.

Bursaries are of the value of £350 per annum, out of which the College Tuition Fee has to be paid ; the amount may be increased to £450 for those with industrial experience. In addition, one or two Senior Bursaries of £600 per annum may be awarded to outstanding men with a minimum of three years' experience in industry.

Applications must be received on or before June 1st, 1954, by the Deputy Registrar, City and Guilds College, Exhibition Road, London, S.W.7, from whom full particulars and application forms may be obtained.

PERSONAL

Mr. W. R. Howard (Member) has taken office as President of the Society of Engineers, which celebrates its centenary this year.

* * *

Dr. A. J. Ockleston (Immediate Past-Chairman of the Union of South Africa Branch) has been appointed Principal of the University of the Witwatersrand.

* * *

Mr. F. M. Bowen (Associate-Member of Council) has become a partner in the practice of Messrs. Scott and Wilson, Kirkpatrick & Partners.

PAPERS FOR PUBLICATION

The Literature Committee would be glad to consider offers of papers for presentation at the Institution or for publication in the Journal.

The following is a summary of the Committee's requirements relating to articles and papers : a copy of the full conditions may be obtained from the Secretary.

(1) Articles must be of an appropriate character, having a bearing upon structural engineering or upon some kindred scientific or constructional subject, and must be approved by the Literature Committee. A short title is an advantage.

(2) Contributions must be original either in subject-matter or in presentation. Articles which have already been published or have been read to other organised bodies, or are carelessly prepared, will not be accepted for publication.

(3) The style of writing will necessarily vary with the individual, but authors are requested to write as plainly and simply as their subject will allow. Papers should be written in the third person.

(4) Where the subject allows, a brief introduction or synopsis should state clearly the purpose and scope of the paper or article, and the author's conclusions or recommendations should be summarised at the end of the paper.

In order to facilitate the indexing of articles for reference, the author will be required in addition to prepare a short precis not exceeding 25 words for inclusion under the title of the paper on the contents page of the Journal.

(5) Illustrations are desirable where they assist in explaining the context or are fundamental to the subject. They should not be used if unnecessary for these purposes. Illustrations may be either line drawings or photographs.

(6) Line drawings must be specially prepared for reproduction on smooth white paper or clear tracing

paper, with heavy main lines and large clear lettering drawn in Indian ink with a mapping pen. Alternatively, the author may submit drawings on one sheet of paper with the relevant lettering on a cover sheet of tracing paper.

The printed page of THE STRUCTURAL ENGINEER is 7 in. wide by 10 in. deep. The drawings, where practicable, should be prepared not larger than twice this size with a view to half-scale reproduction. Unavoidably large drawings which require reduction to one-third size or less, must be specially heavy and with proportionately large lettering for clear reproduction. Ordinary working drawings are not satisfactory.

(7) Where photographs are submitted they should be printed *black on glossy paper*.

(8) MS. typewritten in double spacing should be submitted in duplicate.

Brevity is an advantage and papers should not normally exceed 7,500 words in length.

LONDON GRADUATES' AND STUDENTS' SECTION

The Committee regret to announce that the lecture by Mr. E. M. Lewis on "Welding Construction," advertised for April 13th, 1954, has unavoidably had to be cancelled, and there will now be no meeting of the Section during April. It is hoped that Mr. Lewis will be able to deliver his lecture some time in the near future.

Hon. Secretary : J. F. S. Pryke, B.A.(Hons.), Bushcroft, Slipe Lane, Wormley, Herts.

BRANCH NOTICES

LANCASHIRE AND CHESHIRE BRANCH

The following meeting has been arranged :—

Wednesday, April 28th, 1954

Annual Business Meeting, followed by a film show, in the Reynolds Hall, College of Technology, Manchester, at 6.30 p.m., preceded by tea at 5.45 p.m.

Joint Hon. Secretaries : A. S. Sinclair, A.M.I.Struct.E., 17, The Circuit, Cheadle Hulme, Cheshire ; M. D. Woods, A.M.I.Struct.E., 58, Spring Gardens, Salford, 6.

MIDLAND COUNTIES BRANCH

The following meetings have been arranged :—

Tuesday, April 13th, 1954

Mr. Donovan Lee, B.Sc.(Eng.), M.I.C.E., M.I. Mech.E. (Member of Council), on "Recent Developments in Prestressed Concrete," At the Gas Showrooms, Nottingham, 7 p.m.

Friday, April 30th, 1954

Annual General Meeting, followed by a paper on "Some Factory Building Maintenance Problems," by Mr. W. T. Dudley, A.M.I.Struct.E. At the James Watt Memorial Institute, Great Charles Street, Birmingham, at 6 p.m.

Hon. Secretary : L. A. Firminger, A.M.I.Struct.E., 656, Chester Road, Erdington, Birmingham, 23.

GRADUATES' AND STUDENTS' SECTION

Hon. Secretary : H. L. Bramwell (Graduate), 139, Wood Lane, Handsworth Wood, Birmingham, 20.

NORTHERN COUNTIES BRANCH

The Annual General Meeting of the Branch will be held at the Neville Hall, near the Central Station, Newcastle, on Wednesday, April 7th, 1954, at 6.30 p.m.

Hon. Secretary : O. Lithgow, A.M.I.Struct.E., 4, Stoneleigh Avenue, Acklam, Middlesbrough.

NORTHERN IRELAND BRANCH

The Annual General Meeting of the Branch will be held at the College of Technology, Belfast, on Tuesday, April 6th, 1954. The meeting will commence at 6.45 p.m., and will be preceded by tea at the Overseas League premises, Wellington Place, Belfast, at 6 p.m.

Hon. Secretary : A. H. K. Roberts, B.A., B.A.I., M.I.C.E.I., M.I.Struct.E., "Barbizon," 26, Dunlambert Park, Belfast.

SCOTTISH BRANCH

The Annual General Meeting of the Branch will be held at the Ca'doro Restaurant, Glasgow, on Tuesday, April 13th, 1954, at 6 p.m.

Hon. Secretary : G. Drysdale, A.M.I.Struct.E., "Niaroo," 33, Union Street, Motherwell, Lanarkshire.

SOUTH-WESTERN COUNTIES BRANCH

The Annual General Meeting of the Branch will be held at the Duke of Cornwall Hotel, Plymouth, on Friday, May 21st, 1954, at 7 p.m.

Joint Hon. Secretaries : E. W. Howells, M.I.Struct.E., 10-12, Market Street, Torquay, Devon. ; C. J. Woodrow, "Elstow," Hartley Park Villas, Tavistock Road, Plymouth, Devon.

WALES AND MONMOUTHSHIRE BRANCH

The Annual General Meeting of the Branch will be held at the South Wales Institute of Engineers, Park Place, Cardiff, on Tuesday, May 11th, 1954, at 6.30 p.m.

Hon. Secretary : G. R. Brueton, A.M.I.C.E., A.M.I.Struct.E., 86, The Exchange, Mount Stuart Square, Cardiff.

WESTERN COUNTIES BRANCH

The Annual General Meeting of the Branch will be held at the University of Bristol Geology Lecture Theatre, on Friday, April 2nd, 1954, at 6 p.m., preceded by tea at 5.30 p.m. The Annual General Meeting will be followed by a film show.

Hon. Secretary : E. Hughes, A.M.I.Struct.E., 23, Southdown Road, Westbury-on-Trym, Bristol, 9.

YORKSHIRE BRANCH

The Annual General Meeting of the Branch will be held at the Great Northern Hotel, Leeds, on Wednesday, April 28th, 1954, at 6.30 p.m., and will be followed by a paper by Dr. J. L. Murdoch on "The Quality Control of Concrete."

Hon. Secretary : E. Wrigley, A.M.I.Struct.E., 17, The Drive, Alwoodley, Leeds.

UNION OF SOUTH AFRICA BRANCH

Branch Hon. Secretary : A. E. Tait, B.Sc., A.M.I.C.E., A.M.I.Struct.E., P.O. Box 3306, Johannesburg. During week-days Mr. Tait can be contacted in the City Engineer's Department, Town Hall, Johannesburg. 'Phone: 34-1111, Ext. 257.

Natal Section Hon. Secretary : E. G. Bennett, A.M.I.Struct.E., c/o The Reinforcing Steel Co., Ltd., P.O. Box 49, Merebank, Durban.

Cape Section Hon. Secretary : R. Stubbs, M.I.Struct.E., The Reinforcing Steel Co., Ltd., P.O. Box 2692, Cape Town.

Some Economic Aspects of Single Storey Shed Design

By R. P. Haines, O.B.E., A.M.I.C.E., A.M.I.Struct.E.

Introduction

Of the constructions that fall to the lot of the structural engineer, single storey sheds are perhaps the most prosaic. The traditional medium for such buildings has, for many years, been light latticed rolled steelwork which to a great degree has become standardised by commercial usage.

To fulfil the desire for a more pleasing appearance, reinforced concrete was introduced prior to the war as a competitor to steel in this field. This medium—mainly used in the form of portals and arches—in making a gesture to the æsthetics of design also provided a construction relatively free from dust shelves, gave an unobstructed roof space and involved less maintenance, all factors which mitigate to some extent against the higher first cost.

Since that date the field has been further widened by the introduction of tubular framing, cold-formed construction and prestressed concrete, and the selection of one or other of the media has tended to be a matter of personal choice rather than of relative cost.

The world shortage of materials and the controls applied to their use during and immediately after the war, brought about an artificial situation in which the cheaper materials were not necessarily those most abundant. Conditions therefore became favourable to the adoption of methods of construction which economised in steel, timber and cement, usually at some price disadvantage, and lead to the rapid development of non-traditional forms in rivalry to the established constructions. Some mediums of construction therefore owe something to expediency for their accelerated development and, although in most cases new applications have well justified their sponsors' faith in them, occasionally they have been applied to structures for which they were not pre-eminently suitable.

The engineer is probably now emerging from the phase when availability of material was the primary consideration and greater emphasis must now be given to cost economy while at the same time paying due regard to the sparing use of resources.

The relative economy of materials is a constantly changing factor so that it is not proposed to point out any particular path in design, but to "give the subject an airing," and attempt to show possible paths in which savings might be most profitably made by the designer.

Percentage Cost of Building Components

In considering the economics of design, it is as well to review what proportion of the cost of a shed is expended on each part of the construction, and Fig. 1 indicates the percentages of the total cost that are accounted for by various parts of a typical shed designed by the War Office for storage purposes.

The percentages are based on a structural framework of traditional steel-latticed angle trusses supported on self-standing cantilever joist stanchions. Other types of frame would vary the percentages slightly but not enough to alter the overall picture significantly. It can be seen that the structural frame, which normally suggests itself as the most likely subject for economies, accounts for only about 31 per cent. of the total cost and in consequence a paring down in the weight of steel by ½ lb. per sq. ft. of covered area, which would in any case be difficult to achieve, amounts to a saving of only 10 per cent. of the total steel and would result in an overall saving of only 3 per cent. in the total building cost. On the other hand it is noticeable that the concrete floor, amounting to nearly 20 per cent., is only very little less in cost than the whole of the steel structure and thus presents itself as an equally likely source of economy.

However, the need to save steel is still a major problem so that the greatest attention should be paid to the framing despite the smaller return.

Of the subsidiary items, lead flashings on the glazing account for an out of proportion percentage of the cost and are a significant item in the total cost of the glazing although it must be pointed out that the figures were compiled when lead was at its peak price and more favourable prices are ruling to-day.

In passing, it is noted that since the top and bottom flashings are a constant, irrespective of the depth of the glazing, it may be beneficial to provide extra depth of glazing up to a maximum of 1 ft. over what is required if thereby a run of purlins can be saved.

In considering savings in the framing, it should be borne in mind that the purlins usually account for about 33 per cent. of the total, so that the truss spacing and purlin section adopted will influence the economics of the structure.

Purlins

Purlins are a very fruitful source of steel saving and warrant investigation no less than do trusses in which the paring down of sections achieves possibly a smaller saving.

Purlins are usually fabricated to pass continuously over two spans and are designed on the assumption of partial fixity by assuming $wL^2/9$ as the maximum bending moment (e.g., modulus of section normal to slope $= WL/90$ as given in B.S. 449).

The section of purlin derived from such assumption appears unnecessarily heavy, and it is perhaps time that a change in the construction, or the approach to purlin design, should be made to allow the use of more realistic sizes.

It is notable that in Dutch barn construction, extremely light purlins are made possible by temporarily propping them until the roof sheeting is fixed. Although it cannot be claimed that the pitched roof is analogous to the curved Dutch barn roof, nevertheless some substantial assistance must be derived from the stiffening effect of the roof sheeting.

By reference to Fig. 6 it can be seen that the stresses in purlins calculated on a theoretical basis by the ellipse of stress give apparent values much in excess of the values arrived at by using semi-empirical formulæ such as that recommended in B.S. 449. These theoretical stresses however are never realised nor in fact are the

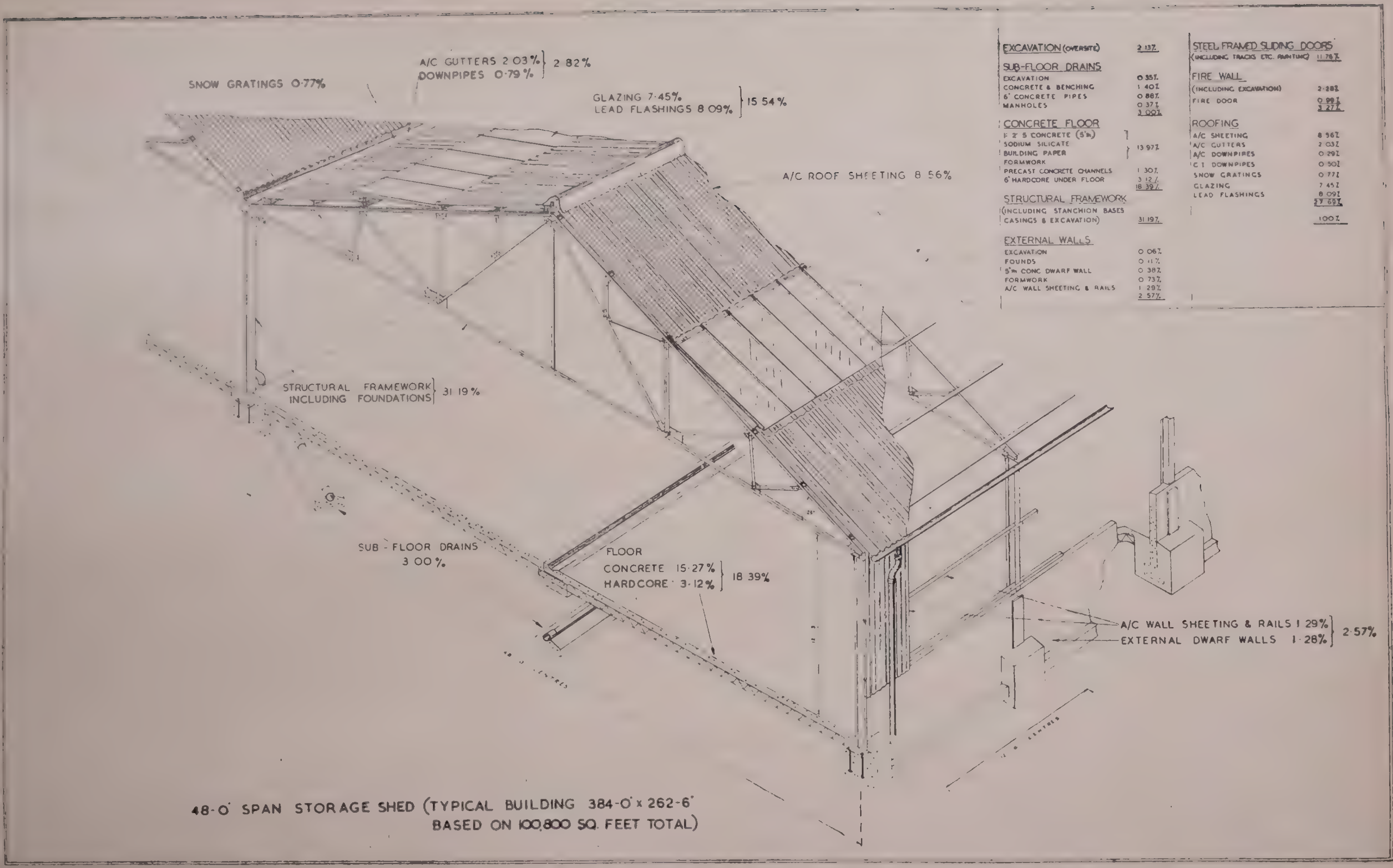

SNOW GRATINGS 0·77%
A/C GUTTERS 2·03%
DOWNPIPES 0·79%
2·82%
GLAZING 7·45%
LEAD FLASHINGS 8·09%
15·54%
A/C ROOF SHEETING 8·56%
STRUCTURAL FRAMEWORK INCLUDING FOUNDATIONS 31·19%
SUB-FLOOR DRAINS 3·00%
FLOOR
CONCRETE 15·27%
HARDCORE 3·12%
18·39%
A/C WALL SHEETING & RAILS 1·29%
EXTERNAL DWARF WALLS 1·28%
2·57%
EXCAVATION (OVERSITE) 2·13%
SUB-FLOOR DRAINS
EXCAVATION 0·35%
CONCRETE & BENCHING 1·40%
6" CONCRETE PIPES 0·88%
MANHOLES 0·37%
3·00%
CONCRETE FLOOR
1: 2: 5 CONCRETE (5"th)
SODIUM SILICATE
BUILDING PAPER
FORMWORK
13·97%
PRECAST CONCRETE CHANNELS 1·30%
6" HARDCORE UNDER FLOOR 3·12%
18·39%
STRUCTURAL FRAMEWORK
(INCLUDING STANCHION BASES CASINGS & EXCAVATION) 31·19%
EXTERNAL WALLS
EXCAVATION 0·06%
FOUNDS 0·11%
5"th CONC DWARF WALL 0·38%
FORMWORK 0·73%
A/C WALL SHEETING & RAILS 1·29%
2·57%
STEEL FRAMED SLIDING DOORS
(INCLUDING TRACKS ETC. PAINTING) 11·78%
FIRE WALL
(INCLUDING EXCAVATION) 2·28%
FIRE DOOR 0·99%
3·27%
ROOFING
A/C SHEETING 8·56%
A/C GUTTERS 2·03%
A/C DOWNPIPES 0·29%
C I DOWNPIPES 0·50%
SNOW GRATINGS 0·77%
GLAZING 7·45%
LEAD FLASHINGS 8·09%
27·69%
100%
48·0' SPAN STORAGE SHED (TYPICAL BUILDING 384·0' x 262·6'
BASED ON 100,800 SQ. FEET TOTAL)

Fig. 1

more moderate values given by the British Standard—for the following reasons : (a) the rigidity of the roof sheeting through its attachments confines the purlin to deflection in a direction nearly normal to the sheeting. Thus bending in the less favourable plane of the theoretical load axis—which would take place if the purlin were unrestricted against rotation and against lateral displacement—is restrained ; (b) the roof sheeting tends to distribute the load to bring into play the lightly loaded ridge and eaves purlins and thus diminish the proportion of load that the intermediate runs of purlins are called upon to carry ; and (c) a better condition of continuity is attained than that which might reasonably be expected.

Fig. 2 shows the actual deflection of a purlin under successive increments of load.

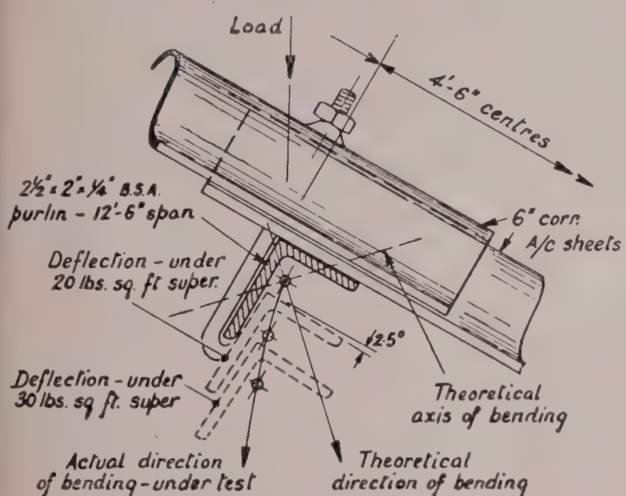


Fig. 2

Were the above not the case it will be seen from Fig. 6 that tubular purlins would have very much better section properties than other sections, and angle purlins could have much improved characteristics by turning the inclined leg up instead of down the slope. No great claim can however be made for the superiority of tubular purlins but angle purlins orientated up the slope show a very slight improvement over normally placed purlins. The advantage of turning purlins up the slope might be more apparent perhaps with R.C. purlins whose self-weight stress—amounting to about 20 per cent. of the total stress—is induced before the sheeting is fixed and able to exercise any restraint on the direction of bending.

Tests have confirmed that the plane of bending is normal, or nearly normal, to the sheeting plane so that the favourable modulus of section $-Z_{xx}$ can be adopted in purlin calculation.

No benefit is derived by using a member continuous over two spans by the " Elastic " method of design as the centre support moment still remains at $wL^2/8$, the same as for a simply supported span. B.S. 449 recognises that the condition is a little better than this and recommends the use of $wL^2/9$; but there seems no reason why the plastic method of design should not be adopted to obtain more realistic sections when purlins are made to run continuously over two spans.

Thus, as shown in Fig. 3, the design moment can reasonably be reduced to $wL^2/11.6$, whilst the plastic *RM* of angle sections may be written as the elastic modulus of section " Z " times the yield stress times a shape factor of approx. 1.86, giving a Z requirement for purlins continuous over an intermediate support of $wL^2/11.6 \times 1/15.25 \times 1/1.86 \times 1.75/1$—where 15.25 tons sq. in. is the yield stress and 1.75 is the load factor on plastic failure for mild steel. Thus $Z_{xx} = wL^2/188$.

If account is taken of the loss of section due to the holes at the truss purlin cleats, the shape factor of the angle at the holed section will be reduced from 1.86 to 1.35 at the central support. To re-establish the balance between the plastic hinges at the span and support, the moments must be adjusted in the ratio $1.35/(1.86+1.35) = 0.42$ and $1.86/(1.86+1.35) = 0.58$. The adjusted moments will be $wL^2/10.7$ and $wL^2/14.75$ as shown in Fig. 3 and the Z_{xx} required when the vertical leg is holed will be $Z_{xx} = (wL^2/10.7)\ (1.75/1.86 \times 15.25)$ or $Z_{xx} = wL^2/173$.

Tubular sections will normally have no holes in them and with a shape factor varying between 1.32 and 1.36 the Z required for grade 13 tube with a yield stress of 13.5 tons sq. in. $= (wL^2/11.6)\ (1.75/1.32 \times 13.5)$ or $Z_{xx} = wL^2/118$.

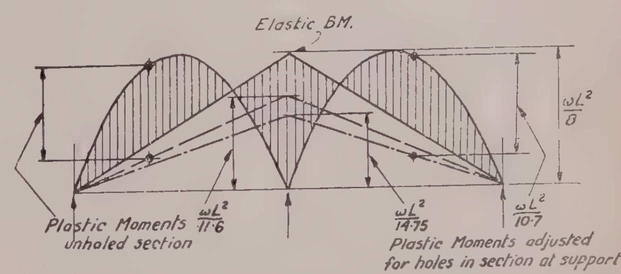


Fig. 3

Cold formed channel sections have a shape factor of 1.19 with holes out, and with a yield stress of 14 tons sq. in., require a $Z_{xx} = wL^2/110$.

M.S. Joist sections will be little affected by a hole at mid-height of the web and with a shape factor of 1.15 give a modulus requirement of $Z_{xx} = wL^2/116$.

In addition the assistance of the ridge and eaves purlins to the more heavily laden intermediate purlins may serve to lower the load carried by these purlins to possible 90 per cent. of its apparent value. No account however is proposed to be taken of this assistance owing to the discontinuity provided if a tier of roof glazing is introduced into a roof slope to make the distribution problematical.

In considering the single span purlins which occur at the ends of the building, there is no doubt that the higher load/deflection properties of the continuous purlins will serve to reduce the proportion of the load shared by the single span purlins with which they alternate. However, if advantage be taken of this assistance, both single and continuous purlins in the end bays would require to be of greater section than those provided intermediately along the building. It is proposed, therefore, that only the single span purlins should be of stronger section so that their deflection characteristics would ensure their carrying their full share of the load.

To conclude, the following section moduli are recommended for steel :—

Purlins continuous over two spans:
- Angle Section $Z_{xx} = wL^2/173$ (Yield 15.25)
- Tubular Section $Z_{xx} = wL^2/118$ (Yield 13.5)
- Cold formed Channel $Z_{xx} = wL^2/110$ (Yield 14.0)
- Joist Section $Z_{xx} = wL^2/116$ (Yield 15.25)

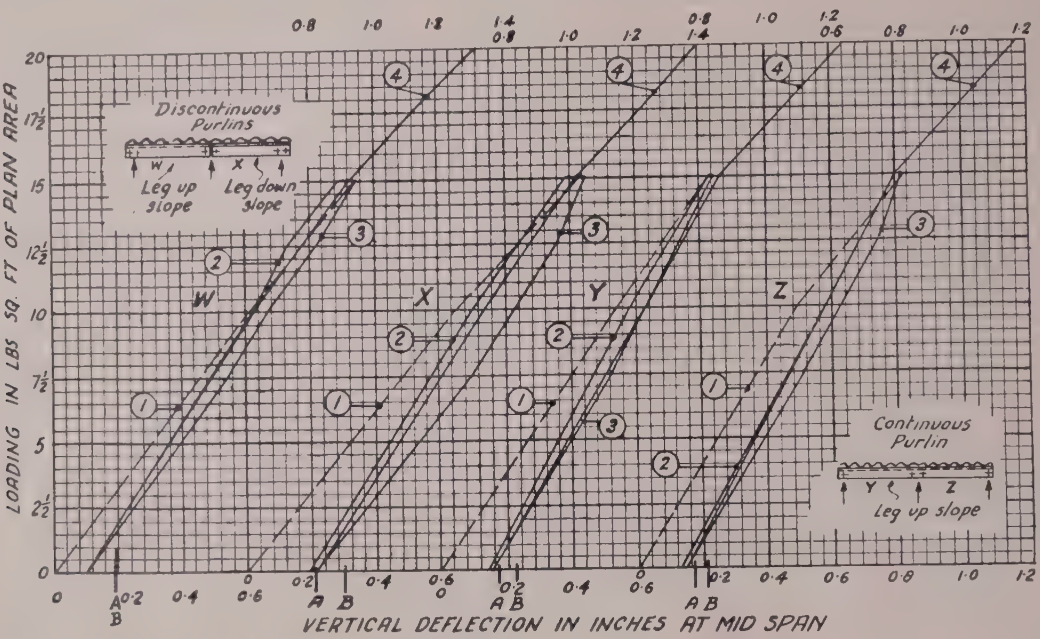
Purlins of single span:
- Angle Section $Z_{xx} = wL^2/130$
- Tubular Section $Z_{xx} = wL^2/83$
- Cold Formed Channel $Z_{xx} = wL^2/77$
- Joist Section $Z_{xx} = wL^2/80$

Calculated on this basis, purlins spanning two spans of 12 ft. 6 in. spaced at 4 ft. 6 in. centres and carrying a superimposed load of 10 lb. per sq. ft., would amply be provided for by a $2\frac{1}{2}$ in. × 2 in. × $\frac{1}{4}$ in. angle section weighing 3.61 lb. per ft. run instead of $3\frac{1}{2}$ in. × $2\frac{1}{2}$ in. × $\frac{1}{4}$ in. by 4.89 lb. per ft. run angle computed by existing methods of design. A saving of about 26 per cent. on the weight of the purlins and 9 per cent. on the weight of the steelwork in the total job could therefore be made.

It may be argued that the smaller section of purlins made possible by this approach might lead to heavy deflections but it is considered that none of the usual cladding materials is so sensitive to flexure as to make this a realistic objection. However, deflection under an actual test with a superimposed load of 10 lb. sq. ft. was recorded as 1/290th of the span, which is a satisfactory figure.

Acknowledgement should here be made to the assistance provided by the Building Research Station who were able to confirm—from earlier studies made at the station—the view that a considerable stiffening effect might be expected to be derived from the cladding on a building frame. Assistance from the B.R.S. in carrying out actual load tests on selected angle purlins was also made available to the War Office, and these tests resulted in the figures shown in Fig. 4 for angles of $2\frac{1}{2}$ in. × 2 in. × $\frac{1}{4}$ in. section. The purlins in the test were spaced at 4 ft. 6 in. centres, spanned 12 ft. 6 in. centres of supporting trusses, and were sheeted with 6 in. corrs. asbestos cement corrugated sheets fastened with hook bolts—finger tight.

The results obtained appear to justify the assumptions made above and lend weight to the adoption of a more rational purlin design. The sheeting used in the test was one year old and apart from one isolated superficial crack, showed no signs of fracture at a load of 3 times the working load. It will be seen from the graph Fig. 4 that in each case after loading the deflection recovery on unloading was within the percentage stipulated in clause 802 of C.P. 113.



A Indicates limit of residual deflection or permanent set allowed in CP 113 acceptance test for first loading.
B Indicates similar limit for second loading.

① Indicates 1st Loading
② Indicates 2nd. Loading
③ Indicates 2nd. Unloading
④ Indicates 3rd. Loading

Fig. 4

A subsidiary test made on $2\frac{1}{2}$ in. × $1\frac{1}{2}$ in. × $\frac{1}{4}$ in. angle showed this section to have unacceptable deflections under 1.75 working load although at 1.5 working load recovery was 90 per cent. In consequence it would appear that as well as satisfying the stress requirements, a limitation in the size of the legs should be imposed. It is therefore proposed that the depth of purlins should be not less than $L/60$ and the minimum breadth $L/75$. These values accord with the $2\frac{1}{2}$ in. × 2 in. × $\frac{1}{4}$ in. angle section tested, and ensure that the Moment of Inertia of other sections will advance roughly proportionately to the cube of the spans on which they are used thus keeping the Span/Deflection ratio nearly constant for all spans. A photograph of the purlin test in progress is shown in Fig. 5.

Notwithstanding the alteration in design assumption proposed for steel sections, there is a considerably wide choice of medium. This choice need not be greatly influenced by the material used in the rest of the structure since all the traditional purlin types can be quite simply connected to trusses whatever the construction of the latter.

Fig. 6 shows a range of purlins of dissimilar materials having approximately the same carrying capacity. The weight of steel and the approximate cost per foo

run erected have been added for comparison. The cost includes for painting steel three coats of paint, and timber one coat of fire retardent paint.

The sections are designed to carry a superimposed load of 10 lb. per sq. ft. measured on plan together with maintenance whereas frequent repainting of steel is necessary. In some cases timber would require the application of periodical fire-resisting coatings. With cold-formed sections some regard must be paid to the possibility of corrosion and although American tests

Superimposed Loading 20 lb. sq. ft. (*Photograph by courtesy of B.R.S.—Crown Copyright reserved*)

Fig. 5

Comparison of Purlin Types – (12′6″ span and 4′6″ spacing, slope 20°)

Purlin Material and Section	M.S. Angle to BS449 — ¼″ metal, 3½″ × 2½″ — Continuous	M.S. Tube Grade 13. — 3″, 10g. — Continuous	Cold Formed Strip — 14g, ¾″, 3½″ × 2″ — Continuous	R.C. — 1½″, 2/¼″ ϕ, 6½″, ½″ ϕ, 4″ — Single Span	Prestressed Concrete — 1¼″, 4/·2″ wires, 4″, 1¼″, 3¾″ — Single Span	Aluminium Alloy AN10B. — ¼″ metal, 4″ × 3″ — Continuous	Timber — 6 13/16″ × 2 5/16″ — Single Span
Weight of Purlin lb. ft. run	4·89	3·89	2·3	15·25	12·2	2·08	3·9
Weight of Steel lb. ft. run	4·89	3·89	2·3	1·15	0·43	–	–
Approx. Cost ft. run fixed	3/1$^{d.}$	3/7^{d}	2/6$^{d.}$	3/1$^{d.}$	3/1^{d}	6/6$^{d.}$	3/3$^{d.}$
Working Stress	10 T/□″	9 T/□″	9 T/□″	20,000 #/□″ steel 1,100 #/□″ con.	2,200 #/□″ comp. 220 #/□″ ten.	6·5 T/□″	1,000 #/□″
% allowable stress-load assumed acting normal to slope (Non symmetrical sections assumed bending in plane of x.x. axis)	93%	94%	95%	100%	95%	100%	94%
% allowable stress calculated on the elipse of stress	159% leg down slope 95% leg up slope	94%	164%	440% leg down slope 105% leg up slope	148%	168% leg down slope 111% leg up slope	183%

Fig. 6

the weight of corrugated asbestos roof covering and a lining (e.g., 15.1 lb. sq. ft. total) in addition to their own self weight. Purlins continuous over two spans have in this case been taken at $WL/9$ and for single spans $WL/8$.

In the choice of a medium, secondary points need attention. Concrete types of purlins need little or no have shown the loss of metal to be slight, in highly corrosive atmospheres the possible loss in such thin walled sections could not be ignored entirely. Consideration of the practical commercial length of timbers and economical cutting will probably limit timber purlins to single span. The same restriction will apply

to prestressed and R.C. purlins, the former because of the loss of section properties that would occur if stressing of both flanges were resorted to, and the latter due to the handling difficulties that would be involved. Thus on all three of these types the benefits of continuity in design would be lost.

If the comparison is extended to take account of the reduced bending moments recommended earlier for purlins carried over two spans, then the picture is materially altered and the section would be amended to those shown in Fig. 7 below.

Purlin Material and Section	M.S. Angle	M.S. Tube Grade 13.	Cold Formed Strip	R.C.	Prestressed Concrete	Aluminium Alloy A.W.10B.	Timber
End Condition	Continuous over 2 spans (Holes out)	Continuous over 2 spans	Continuous over 2 spans (Holes out)	Single Span	Single Span	Continuous over 2 spans (Holes out)	Single Span
Section Modulus for Plastic design where applicable	$wL^2/173$	$wL^2/118$	$wL^2/110$	–	–	$wL^2/88$ *	–
Z. Required	0·34	0·5	0 53	–	–	0·664	–
Size of Section	$2\frac{1}{2}'' \times 2'' \times \frac{1}{4}''$	$2\frac{3}{8}''$ ⁰/D 9g.	$3'' \times 2''$ 14g.	$6\frac{1}{2}'' \times 4''$	$4'' \times 3\frac{3}{4}''$	$3\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{1}{4}''$	$6\frac{13}{16}'' \times 2\frac{5}{16}''$
Weight ft. run Total	3 61	3 43	2 162	15·25	12 2	1·77	3·9
Weight ft. run Steel	3·61	3·43	2·162	1·15	0·43	Nil.	Nil.
Cost per foot run f/d.	2/6½d.	2/9d.	2/5d.	3/1d.	3/1d.	5/8½d.	3/3d.

* A reduced yield value has been adopted to give the same factor on the ultimate stress as steel

Fig. 7

It would appear on this comparison, that the cold-formed and M.S. angle purlins are superior to other types, and tubular purlins do not make as good a showing as would be expected. It is perhaps fair to point out that the use of prestressed concrete for purlins does not do justice to this medium since its self weight is comparatively low and more than balanced by the loading condition of upward suction wind when used on the lee slope. Additionally—unless a rectangular section is used, mould costs require a good repetition in order that the price shown can be realised.

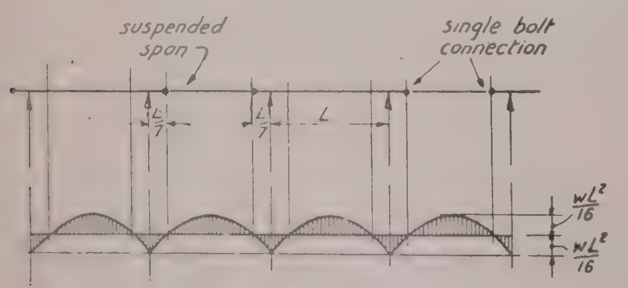


Fig. 8

If, for some reason, it is not desired to work on the plastic collapse method, or if truss spacing is so large that supply difficulties will be encountered in providing purlins of a length sufficient to stretch over two spans, then alternative constructions may be adopted to improve the value of the induced bending moments.

One such construction which has found a good deal of favour on the Continent, uses a suspended span to give a bending moment maximum of $\frac{wL^2}{16}$ and a required Z_{xx} of $\frac{wL^2}{160}$ for mild steel. The diagram Fig. 8 shows this well-known arrangement.

A second construction, which is a little more costly, achieves the same result using single span lengths of purlins and welding their butting ends at the junction over the truss, with a full strength weld. Until site welding becomes less costly this construction will however be hard to justify.

The use of either of the above constructions gives a favourable deflection which is only about one-fifth of that resulting from the use of simply supported purlins of single span.

Spans and Spacing of Trusses

There has been a growing tendency in modern design to favour large spans despite the fact that for many "Users" no great benefit is derived. This trend is possibly due to the advent of new materials which can be exploited more favourably when spans are great ; but in general a decrease in span will be accompanied by an economy in the use of materials, provided that the span does not become so small that reduction in the component parts of the truss is restricted by consideration of practical minimum sizes.

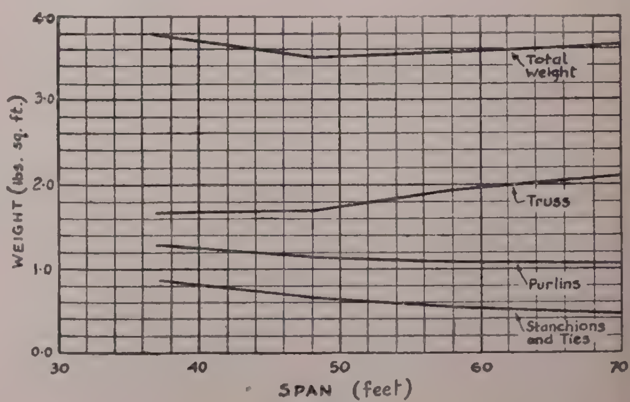


Fig. 9

The total length of the members in a framed latticed truss varies lineally with the span whilst the section of the members varies to a somewhat lesser degree so that

the weight of the truss is roughly proportional to the ratio of the spans to the power of 1.8, and its weight per sq. ft. of covered area increases a little less than proportionately to the increase in span.

To illustrate this point, conventional riveted steel angle latticed trusses of 48 ft. span spaced at 12 ft. 6 in. centres weigh 915 lb., whilst similar trusses of 58 ft. span weigh 1,310 lb. The ratio $(58/48)^{1.8} \times 915$ gives 1,290 lb., which is very near the actual weight of 1,310 lb. Similarly, 68 ft. span trusses give a calculated weight of 1,710 lb. against an actual of 1,655 lb.

The weight of purlins is nearly a constant for any particular spacing of trusses and is independent of the truss span except for the variation caused by the greater

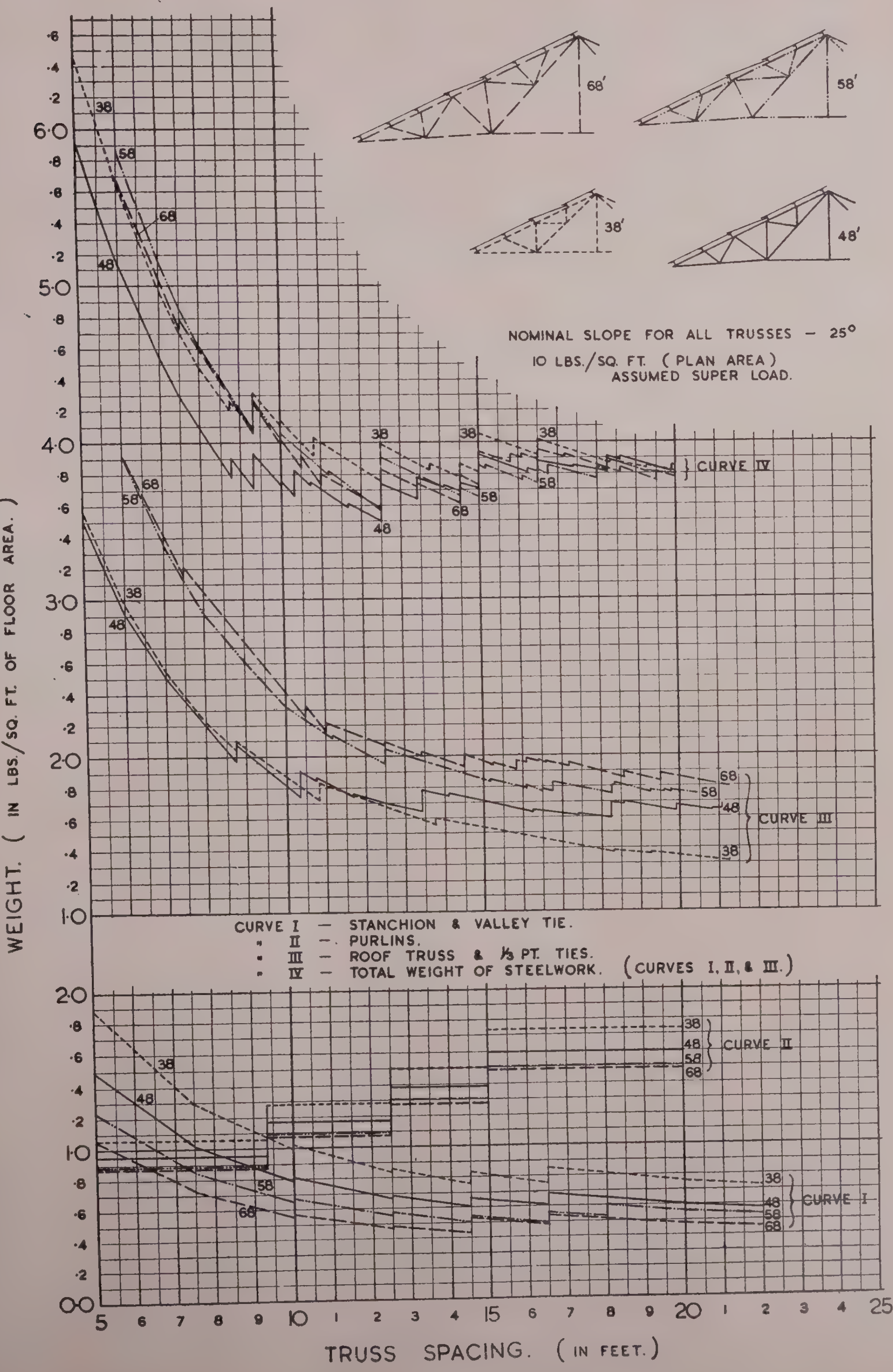


Fig. 10

incidence of double purlins at the ridge and valley on smaller spans.

The graph Fig. 9 based on 12 ft. 6 in. centres of riveted trusses shows that weight saving is progressive as the span decreases until, as a greater number of members reach their minimum practical size, the reduction loses its significance and is swallowed up by the extra weight caused by ridge and valley purlins. The optimum span is between 45 ft. 0 in. and 55 ft. 0 in. for this type of truss, although there is surprisingly little increase in weight at spans outside the range.

A truss spacing of 12 ft. 6 in. is found to be favourable for spans of the order of 40 ft. to 70 ft. At this spacing the bulk of the secondary internal truss members are of nominal minimum size so that an increase in the spacing will lead to only very slight increases in the truss scantlings. However, even if this small increase is ignored, the effect of using the next larger angle section for purlins more than offsets the saving made by the lesser total number of trusses and stanchions needed.

The relationship between truss spacing and weight of steel per sq. ft. for buildings of 38 ft. to 68 ft. span with an eaves height of 12 ft. 6 in. is graphically represented in Fig. 10. The weights include all bolts gussets etc., and angle glazing cills.

It can be concluded from the foregoing graphs, that for buildings with normal eaves heights constructed with light steel latticed trusses, the most favourable span of truss lies in the range 45 ft. to 55 ft. and the most satisfactory spacing of these trusses is 12 ft. 6 in. For spans over 70 ft. a slightly increased spacing is indicated.

Although these figures might vary slightly for other materials of construction the same sort of order might be expected to [illegible].

A 48 ft. span truss spaced at 12 ft. 6 in. centres was adopted for War Department storage sheds and the original design, using $3\frac{1}{2}$ in. by $2\frac{1}{2}$ in. by $\frac{1}{4}$ in. angle purlins to B.S. 449, gave the not unfavourable weight of 3.86 lb. sq. ft. of floor area for the total steelwork including stanchions, for an internal bay. Fig. 11 shows this construction during erection. A redesign using the $2\frac{1}{2}$ in. by 2 in. by $\frac{1}{4}$ in. angle purlins recommended in this paper improved the weight to 3.50 lb. sq. ft.—this accords with the weight shown in the graph Fig. 10.

The foregoing weight comparisons have been based on the use of valley stanchions at each truss. As the economics of roof construction dictate a truss spacing of about 12 ft. 6 in., if reasonably unrestricted movement between one span and that adjacent to it is required, open spaced valley stanchions with valley beams may have to be introduced to carry the intermediate trusses. Except in very high buildings or in buildings in which gantry cranes are to be installed, the inclusion of valley beams leads to a loss of economy in the structure and, except where required for the foregoing reasons, they should be avoided.

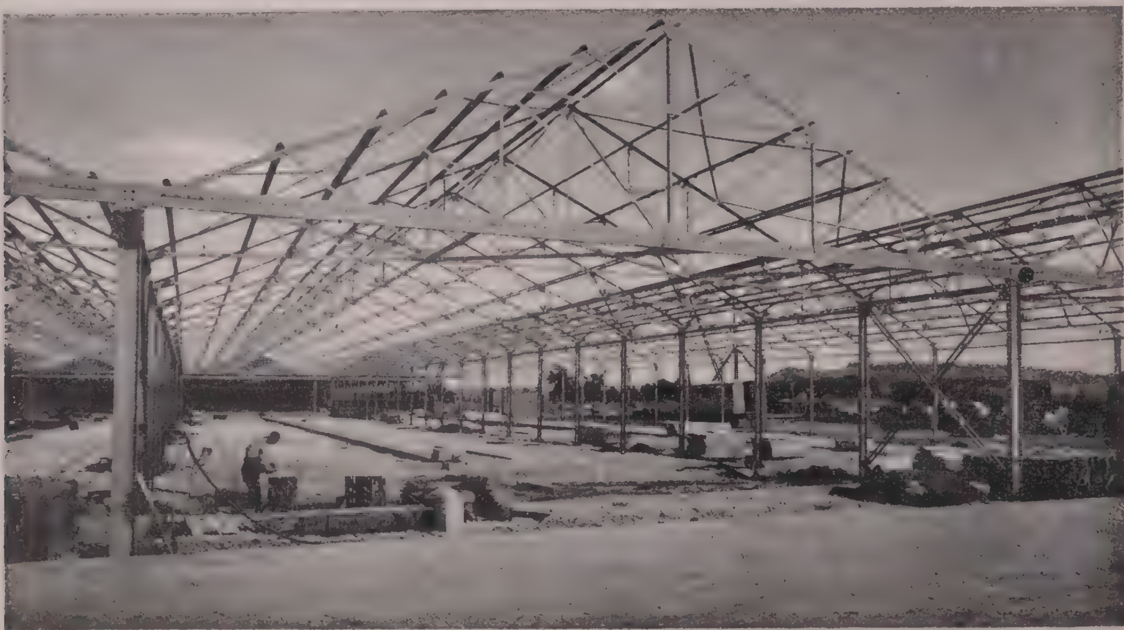


Fig. 11

The additional steel in valley beam construction is about 0.65 lb. per sq. ft. of floor area on a normal steel frame of low eaves height with trusses at 12 ft. 6 in. centres and valley stanchions at 25 ft. 0 in. (e.g. an additional 18 per cent. in the total steel weight).

Where valley beams are essential, some saving in section and weight is possible by the use of connections which allow the design to be based on complete continuity ; or by the introduction of knee braces—where these are permissible—to break the unsupported length of the beams. Alternatively, the bending moments may be reduced by the use of the suspended span. A practical example of the latter system is illustrated in the reinforced concrete construction shown in Fig. 12.

Portal Frames

The employment of welded steel joist sections or reinforced concrete members normally dictates the use of portal frame construction. As to choice of type, the diagrams in Fig. 13 show the bending moments induced in a frame under varying conditions of end fixities. The total area of the bending moment diagrams has been chosen as the yardstick for comparison and a percentage efficiency has been derived from these figures. This assumption is not quite true but is not thought to favour any particular type of portal and so is felt to be reasonable for comparative evaluation—especially where the

strength of the member can be varied by modifying the section profile to accommodate points of high moment.

Assuming that the relation of column height to span is representative of the proportions of the average portal, it would appear that the tied rafter designs-cases 1 and 2—would normally give the most economical usage of materials and would be at least 32 per cent. better than the popular three-pin frame given in case 5. The fixed base portal-case 3 is nearly comparable to the tied rafter design but it is only suitable for use where the ground

for self-weight stresses during erection only—reliance being placed on the buttressing effect of adjacent spans to balance the horizontal thrusts due to the weight of the cladding and of the snow load.

For welded steel joist portals, no variation in the section along its length is really feasible within about 1 ft. 6 in. of the base, and each side of the haunch, and apex joints, so that the maximum bending moments beyond these points become the criteria for design for this type of construction.

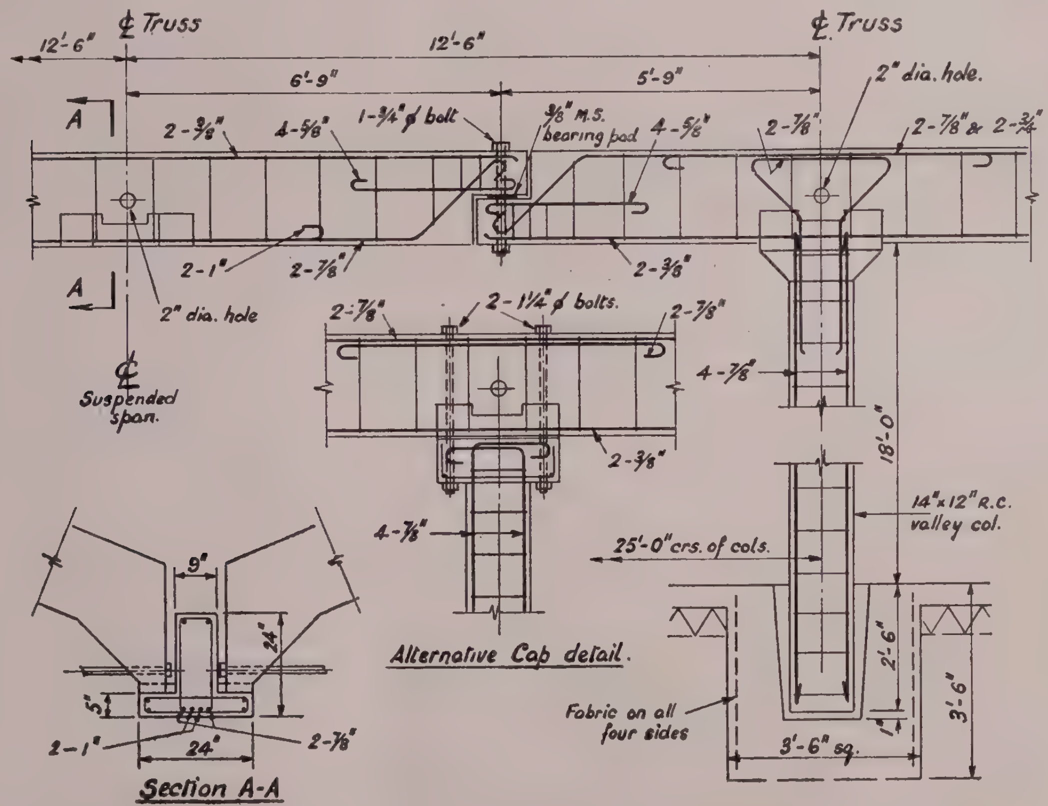


Fig. 12

conditions justify the assumption of rigid foundations. Also in comparing the two, no account has been taken of the greatly increased size of foundation required with the rigid portal, or with the difficulties attendant on casting this type of frame *in situ*. Although the rigid portal frame could be made up of factory-made components by the introduction of scarfed bolted site joints at or near points of contraflexure, rather cumbersome and bulky components result which are in no way comparable to the lighter members required in the tied rafter designs—particularly the three pinned type which lends itself most readily to factory production and delivery by road or rail. These latter types also have the advantage that they can readily be used for multiple span buildings which incorporate valley beam construction.

Two features of the three pinned tied rafter design shown are perhaps worthy of note. One is the deliberate offsetting of the apex pin and tie from the calculated neutral axis of the rafter to induce a reverse moment, thus utilising the eccentric action of the direct thrust to reduce the maximum moment by 30 per cent. The other is the possible further economy that could be effected in multiple span buildings by reducing the size of the lower tie rod in the internal bays to that needed

The significant values for comparison of the five cases would then be as follows :—

Case 1 B.M. = 33,750 lb. ft., e.g., 21,750 lb. ft., plus an allowance of, say, 12,000 lb. ft. for the equivalent moment of the tie steel reduced to normal lever arm depth of the rafter.

Case 2 B.M. = 35,400 lb. ft., e.g., 20,400 lb. ft., plus 15,000 lb. ft. allowance for the tie steel.

Case 3 B.M. = 42,000 lb. ft.

Case 4 B.M. = 53,000 lb. ft.

Case 5 B.M. = 71,000 lb. ft.

This then shows the tied rafter designs to compare even more favourably than on the comparison of bending moment areas.

It should also be noted that for cases 1 and 2 the columns can be of smaller section than the rafters, whereas in the frames, cases 3, 4 and 5, the rafter moment is continued round the haunch to require the same section in the column at this point.

It is not contended that cost will be proportionate to the values calculated for the moments set up in the

frames, but it is maintained that the trend shown will be reflected in the price of the structure in any chosen medium.

The tied rafter designs thus appear to be the best form of construction when using reinforced concrete or steel joist construction designed to the elastic theory, and is so poor as to cast grave suspicions on the ability of the foundations to develop the fixity required for the other forms of construction, or in areas where the ground is subject to subsidence.

For intermediate ground conditions sufficient passive resistance could be easily developed to satisfy the small

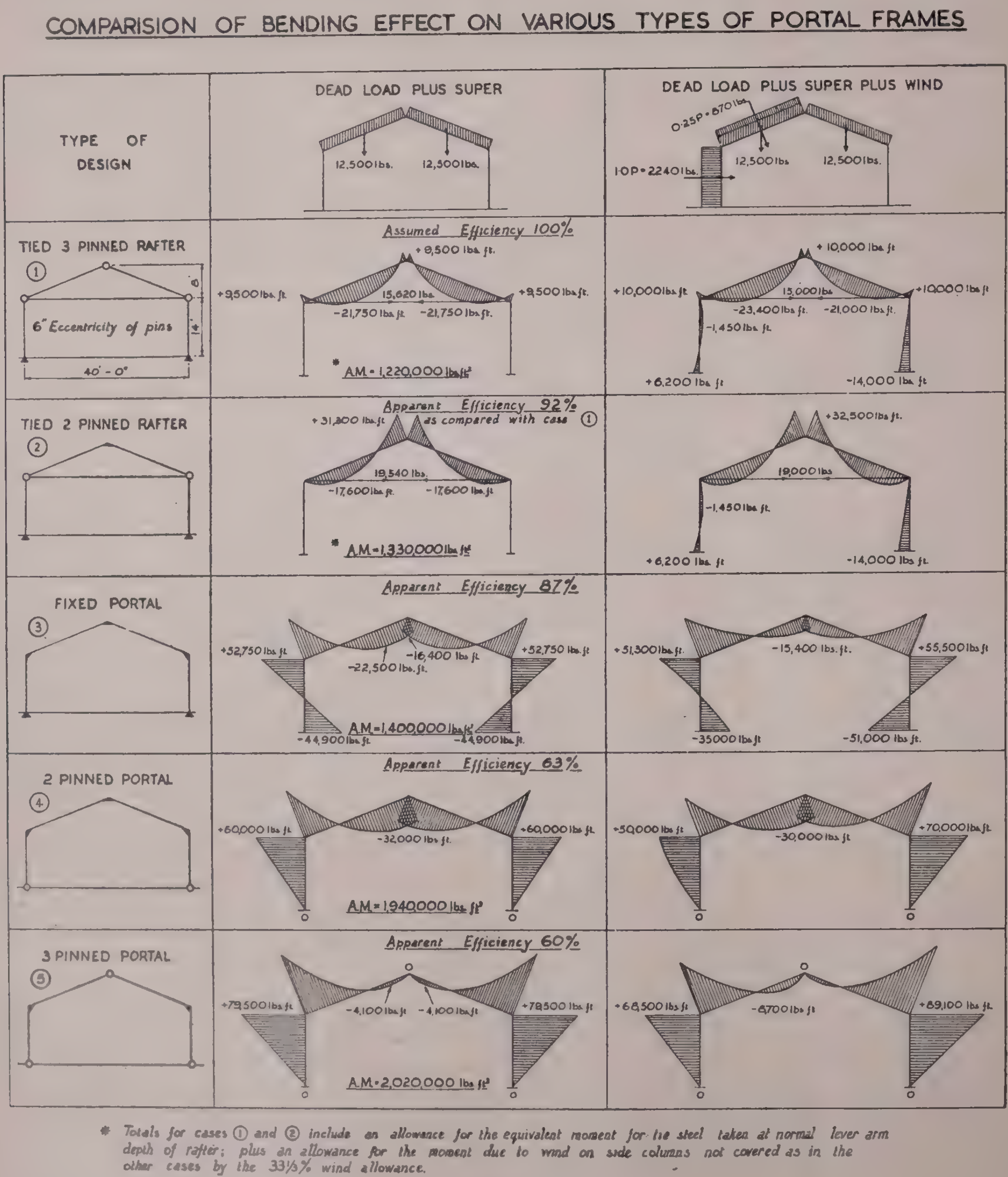


Fig. 13

of the two, the three pinned type appears the cheapest and simplest to construct.

The other types have special features which commend them for particular purposes. For instance, where the roof space above tie level can be usefully employed, the open portal type of frame might be worthy of consideration. Also the three pinned arch construction has the advantage that it is able to accommodate differential settlement and also requires only nominal foundations. It might therefore be justified where the type of ground fixing moment required by the self-standing cantilever columns of the tied rafter design. This structure also has the advantage that it is more flexible than the portals and small movements of the column foundations would not induce the high secondary moments attendant on movement in a rigid portal frame.

From the economic standpoint some materials are less efficient than others. The strength weight ratio of the material used will determine the self-weight of the structure so that frames which by their nature require

the use of a material with a low strength/weight ratio may be handicapped thereby. The self-weight of the roof principals of a single storey shed of steel rafter construction amounts to about one-tenth of the total design load whereas, to carry the same total load, a reinforced concrete structure of similar construction would have members twice the depth weighing 4.6 times the weight of the steel beam doing the same job.

If the self-weight of the structure therefore happens to be an appreciable part of the total load being carried, then the self-weight would again need to be increased in order to carry itself. Ultimately it would take about 9 lb. per sq. ft. of floor area of an R.C. rafter construction to carry its own weight plus 15.5 lb. per sq. ft. of superimposed load (e.g., 24.5 lb. per sq. ft. total), whilst only 1.6 lb. per sq. ft. of steel would be required to carry—in the case of steel—the total of 17.1 lb. This extra weigh to be carried would alter the comparison of moments in favour of frames employing high strength/weight ratio materials.

Prefabricated components of materials with low strength/weight ratio will be at a disadvantage where the distance from the site to the factory is great, and in addition they will be slower to erect and will require more erecting equipment. To illustrate this point, the total weight of the framing of a steel shed of 100,000 spare feet area is 210 tons, whereas the comparable weight of reinforced concrete units for the same job is 850 tons.

Column 7 of table shows the weights of members of various materials after adjustment of depth—carrying the same superimposed load. Column 8 of the table shows the approximate relative costs.

However, as shown in the table, despite the heavier weight of material, reinforced and prestressed concrete compare reasonably with steel if rafter type frames are used and elastic design methods are adopted.

In summing up it might be said that if there is no restriction as to type then the tied rafter design is the most favourable construction for portal type frames designed on elastic principles, and steel is the most favourable medium. Reinforced and prestressed concrete will show a good steel economy whilst being nearly competitive in price when the cost of maintenance is taken into account, provided that jobs are of sufficient size to allow the mould costs to be spread over a large number of repetitive assemblies.

Plastic Design

The comparisons would not be complete without considering the plastic method of design. Since failure in steel undoubtedly takes place initially by plastic distortion leading to stess re-distribution it would appear logical to base design on this premise.

Generally, the greater the disparity between hogging and sagging moments in a frame or member designed on elastic principles, the greater the advantage gained by using the plastic approach, and by the introduction of redundancies an even better utilisation of material is possible.

The diagrams in Fig. 14 show the elastic bending moments for dead plus snow loading, with the relative equivalent plastic bending moment diagrams for joist sections, superimposed. The diagrams illustrate that a

TABLE 1.—Comparison of Self Weight of Rafters of Various Materials

Material	Type of Section	Resisting Moment in in. lb. taking $b = \frac{d}{2}$ Qd^3	Weight of Section in lb. per ft. run- taking— $b = \frac{d}{2}$ Cd^2	Relative depth of Sections in each material compared with aluminium as unity when Super. load is 15.5 lb. per sq. ft. and aluminium section self weight is 0.75 lb. per sq. ft. of covered area $d_x = \sqrt[3]{\frac{Q_a}{Q_x}\frac{(15.5+W_x)}{(16.25)}}$	Strength/Weight ratio adjusted for depth and related to Aluminium as unity $R = \frac{Q_x\ d_x\ C_a}{Q_a\ d_a\ C_x}$	Self Weight of member in lb. per sq. ft. of covered Area $W_x = \frac{W_a(15.5+W_x)}{R(15.5+W_a)}$	Approx. Unit Cost per lb. of material erected	Relative cost of members in each material
Aluminium Alloy(A.W.10B) (f = 6.5 tons)	Joist	$425\ d^3 = Q_a d^3$	$0.11d^2 = C_a d^2$	$1.0 = d_a$	1.0	$0.75 = W_a$	3/6d.	2/7½d.
Timber Commercial Softwood (f = 1,000 lb.)	Rectangle	$83.5d^3$	$0.125d^2$	1.79	0.305	2.76	-/8d.	1/10d.
Mild Steel (f = 10 tons)	Joist	$600d^3$	$0.285d^2$	0.91	0.49	1.6	-/7.8d.	1/0½d.
Prestressed Concrete (fc=2,000 lb.)	Joist	$142.5d^3$	$0.315d^2$	Post tensioned—1.42	0.166	4.32	—	—
				Pre-tensioned—1.56	0.183	5.23	-/2.7d.	1/2d.
Reinforced Concrete (fc = 1,000 lb.) (ft = 20,000 lb.)	Joist	$74.5d^3$ Φ	$0.315d^2$	2.05	0.126	9.0	-/1.54d.	1/2d.

NOTE: (Φ) The depth d for reinforced concrete is the overall depth allowing for the cover to the tensile reinforcement.

reduction in moment of between 25 per cent. and 33 per cent. is made possible by the assumption that the formation of a plastic hinge at the highly stressed apex will cause a "throwing-off" of further moment—due to

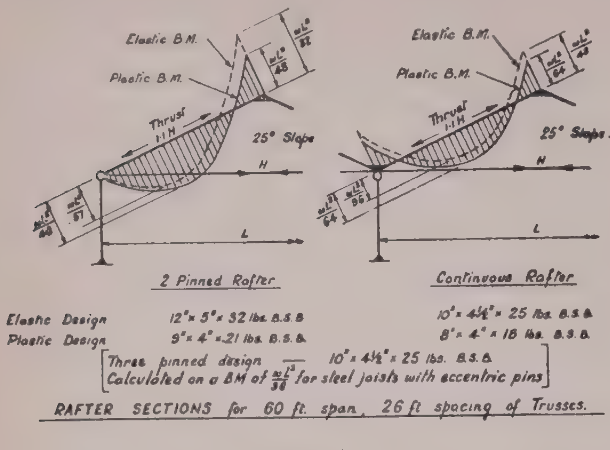


Fig. 14

...ve load increments—to the moment at quarter ...an until all points of high moment reach equality at plastic collapse. The plastic moments shown allow a load factor of 1.75 on the collapse load to make the two diagrams comparable.

The relative sections required for a 60 ft. span frame are also shown.

There are two possible objections to the practical application of plastic design: (a) full benefit is not realised unless all joints of a frame are designed to be continuous, so that the full section should be developed at all site connections. This may prove expensive with the present cost of site welding. (b) The design produces slender frames which, being flexible and whippy, offer little support during erection. Temporary strutting to a large measure may have to be resorted to.

Fig. 15 shows a rafter construction detailed by the War Office to a plastic design prepared by Professor J. F. Baker in which full advantage is taken of continuity at the valley springing as well as at the apex. The great merit of the construction is that fully continuous purlins are used to permit a wide truss spacing without any increases in the purlin weight over that which would be required with conventional designed purlins with their narrower truss spacing. A further saving is accomplished in the truss frames themselves, and a still greater economy is made possible by the omission of valley beams.

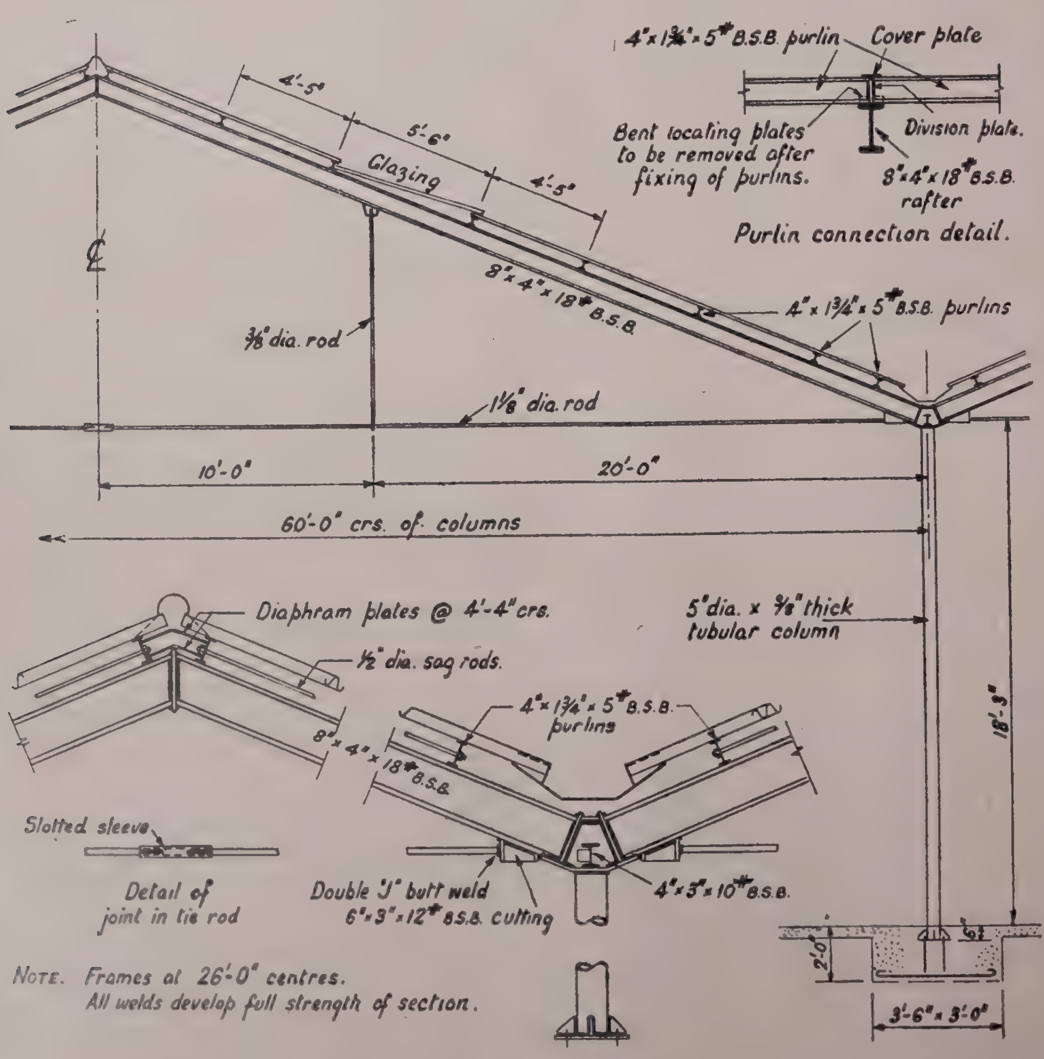


Fig. 15

These would have been required in this particular instance to give reasonable unrestricted access between bays if a narrower truss spacing had been used.

On the face of it the purlins appear exceedingly slender, but sag rods take care of the lateral deflection

and the continuity over the support of course reduces the vertical deflection to 1/5th of that present with a similar section purlin simply supported at the ends.

The lower tie rod of the truss was included as a precaution against spreading and creep and as a further precaution it was proposed to induce a pre-tension in this tie and a consequent slight initial reverse moment in the apex joint. The specification provided for temporary "finding bolts" to be provided to assist location and site welding.

The comparative weights of steel in lb. sq. ft. of floor area in a building of this construction and in a building of conventional riveted angle construction are of interest and show that, providing the cost of welded steelwork

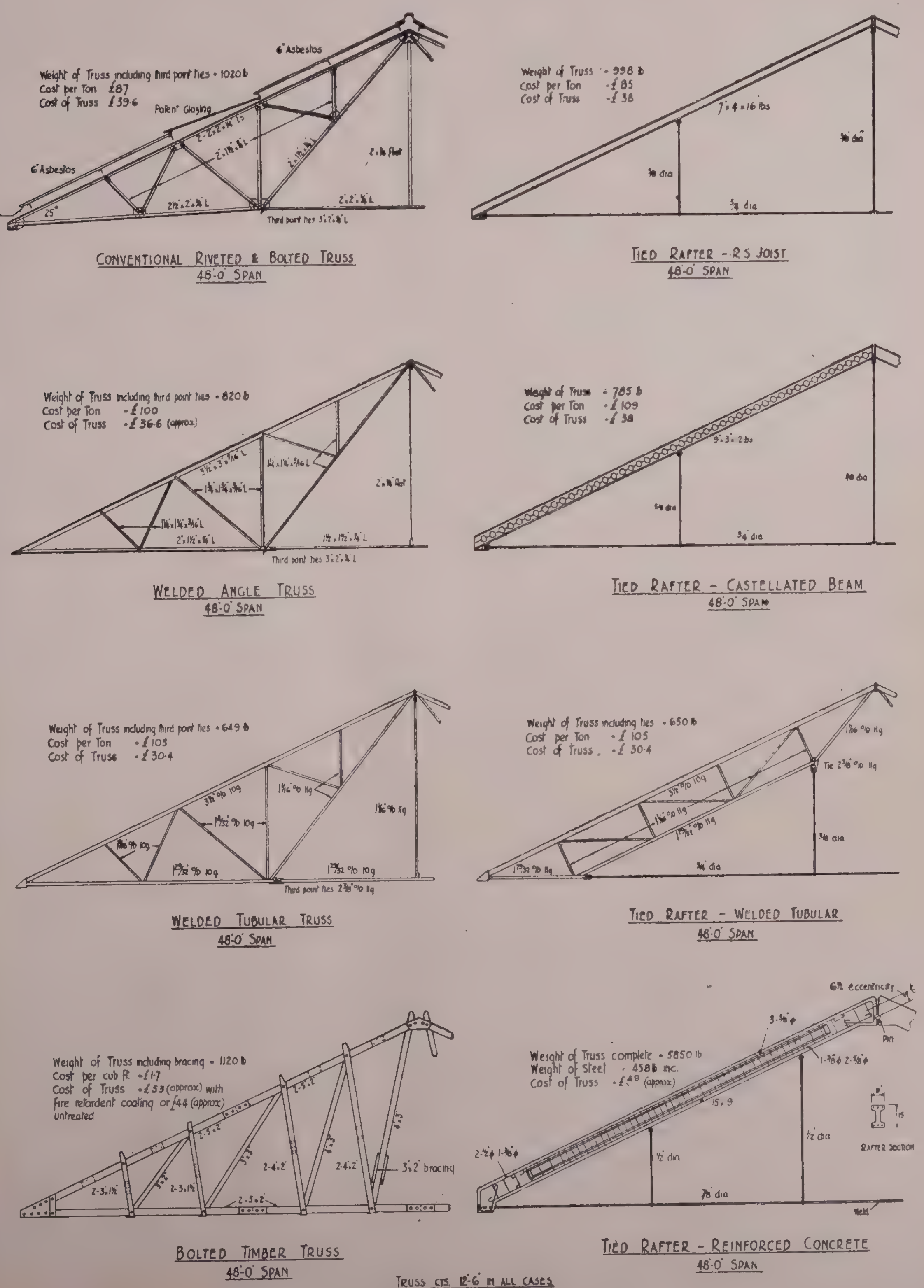


Fig. 16

approaches £120 per ton, economy would be achieved by its use.

		Welded Plastic Design	Conventional Riveted Design (B.S. Purlins)
60 ft. Span	Internal Bay	2.96	4.80
	Whole Building	4.35	6.255
48 ft. Span	Internal Bay	2.58	3.86
	Whole Building	3.02	4.69

This price level of £120 per ton is of course very high, and might reasonably be expected to be realised by contracting firms with experience of site welding. For small jobs, where it might be advisable to avoid welding on site, it is possible that some compromise such as a two pinned rafter with a deepened and bolted rigid apex joint to obviate site welding, may provide the right answer. The weight saving of course will not be so spectacular. A purlin system incorporating pins close to the points of contraflexure as shown in Fig. 8, may also provide a simplified construction for these members.

Trussed Frames

It has been shown in the earlier comparison, that the tied rafter types of frame have better B.M. characteristics than other portal types, and further secondary triangulation to replace the bending moments in the comparatively shallow rafter sections by a trussed system of members carrying direct forces, might be worthy of investigation.

No great merit can be claimed however for a conventional riveted angle truss over a steel R.S.J. rafter frame mainly because of the poor utilisation of stress in the trussed construction. Strut stresses are much reduced by their high slenderness ratio ; ties, with holes and half the outstanding leg non-operative, are only about 60 per cent. efficient ; and in addition the further reduction occasioned by the necessity to adopt practical minimum sections to provide legs of bolting width and sections sufficiently robust for handling and erection, has a not inconsiderable effect.

Welded trussed rafters and welded trusses are more favourably disposed and suffer less from stress losses. Their ties are about 80 per cent. efficient and their minimum sections are unrestricted by any necessity to have legs of bolting size.

The employment of tubular welded construction permits concentric loading of members and the full area of ties to be developed. This, together with the improved strut characteristics of tubes, allows a reasonable economy over the other forms of trussed frames.

Comparisons have been made in Fig. 16 of steel trusses and trussed rafters of various forms. It will be seen in this example that there is little to choose between the trussed rafters and the corresponding triangulated trusses from the point of view of weight alone. However, provided that there are no other considerations, the choice of construction must be governed by cost and the designer should therefore consider whether the more expensive fabrications can contribute sufficient weight saving to justify their employment.

An attempt has been made to cost the constructions illustrated but with the normal fluctuations in prices, costs are difficult to define accurately, and the figures, although bearing reasonable authenticity, may be varied from case to case to modify the small margins of difference shown.

Costs of steel trusses include erection and painting three coats of red lead.

The picture could be distorted by considering comparisons of one single span only. In particular, joist rafters will suffer from the limited number of sections available from which to select. Fig. 17 therefore shows a weight comparison of rolled steel joist rafters and riveted angle trusses at spans ranging from 38 ft. to 58 ft. The former construction is shown to be slightly superior at short spans whilst the latter gains at spans of 60 ft. and over. In the mid range of spans there is little to choose between the two, the maximum weight variation being of the order of $7\frac{1}{2}$ per cent.

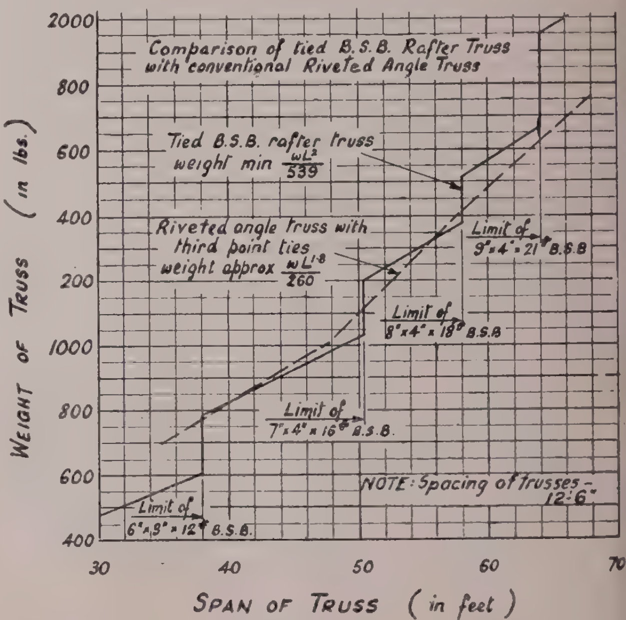


Fig. 17

Basically, however, the comparison made in Fig. 16 will be fairly representative of all spans and the conclusions drawn from this comparison might be accepted as typical.

The most popular construction by usage is the conventional riveted steel angle truss which is accordingly the most readily available, and since the majority of fabricating shops are laid out for this type of work the welded constructions have as yet not been very widely adopted. The foregoing examples would appear however to indicate that because of their low weight combined with low cost, far greater emphasis should be placed on welded tubular and cold-formed assemblies than hitherto.

The estimates of cost have been based on large buildings where the amount of repetition shows the welded job in a more favourable light than on smaller jobs where the cost of jigging may be spread over a smaller number of assemblies.

The maintenance element should also not be overlooked and the need to paint steel periodically must be offset against the higher initial cost of materials with a more durable natural surface. The painting cost would amount to about 6 per cent. of the initial cost of steel for every six years of its life.

There is of course a limit to the employment of the tied rafter design in steel, and for wind forces in excess

of about 75 m.p.h. the dead weight of the steel roof construction will be overcome to allow a stress reversal in the lower tie rod to be induced. The same is not true when reinforced concrete is used since with this medium the far larger weight would not be balanced within the range of practical maximum wind pressure.

In the foregoing comparisons, the answer is obviously influenced by the minimum sizes adopted for each construction. Two inch bolting legs and $\frac{5}{8}$ in. dia. bolts have been assumed minimum for the bolted and riveted angle truss; the tubular truss has been restricted to 10 gauge main members and 12 gauge secondaries; and $1\frac{1}{4}$ in. × $1\frac{1}{4}$ in. × 3/16 in. angle has been taken as the minimum for welded angle trusses. These sections may be considered to be somewhat light but it is of interest to note that welded trusses in Belgium have frequently been used with $1\frac{1}{4}$ in. × $1\frac{1}{4}$ in. × $\frac{1}{8}$ in. angle section.

The comparison includes an example of the use of castellated steel beams. These beams give a higher section modulus at a lower weight than normal rolled steel joists, and are suitable for use where the member is effectively stayed against lateral buckling and the shear is not excessive. The weight of these beams may be as low as 75 per cent. of the weight of joists of comparable strength and the basic cost of materials per ton is about 40 per cent. more than normal joist section.

Trusses made from aluminium sections have not been considered because of the high cost in short span structures where the lower self-weight of aluminium is unable to make a significant contribution. It may be that under certain conditions the corrosion resisting properties of some of the purer alloys of aluminium may recommend their use for work in corrosive atmospheres.

Undoubtedly, cold-formed sections provide by far the lightest steel structures. The desirability of applying a dipping treatment of phosphate before painting to improve the corrosion resistance of the metal may offer a restriction to the size of single assemblies. Compared with the constructions in Fig. 16 a 48 ft. span truss of cold-formed sections will weigh together with its third point ties 512 lb. The rate for this work erected will be of the order of £145 per ton allowing for phosphatising, priming with zinc chromate and painting two coats; thus giving a figure of £33 for the truss and ties.

Cold-formed trusses, however, really come into their own at short spans where other constructions are forced to use heavier sections than required by stress considerations.

In examining trusses, the following points are worthy of note :—

As a general rule—although the difference is not large—it will be found advantageous to increase the triangulation of a truss rather than incur secondary bending in the rafter back.

In the design of truss back rafters, it is for consideration whether the argument for reducing the allowable bending stress for lateral buckling as well as reducing the allowable strut stress for crippling, is tenable. If the critical point in a rafter is under a purlin supported between the node points of the truss, then at this point it is effectively held laterally by the purlin and an $\frac{L}{B}$ reduction appears inappropriate; if—as is usually the case—the worst fibre stress occurs at the node support, then any $\frac{L}{K}$ reduction at this point seems unduly severe.

Certain benefits would appear to be derived from the introduction of knee braces in trusses but their use is restrictive on headroom and any saving made in the stanchion section and truss main tie is offset by the larger truss sections required round the knee.

The greater emphasis put on wind suction effect in modern design has almost resulted in the abandonment of flats in truss construction. This trend has gained strength by the fact that the use of flats has sometimes been accompanied by undesirable snaking in so-called tie members, probably due to incorrect shortening of members in the templating shop. However, for buildings in sheltered areas where wind pressures are not excessive, or where high winds are encountered but due to the absence of openings in the cladding internal pressure is not applicable, there seems no reason why the practice of using flats or round rod ties should not be more generally resorted to in riveted trusses. A weight advantage of some 15 per cent. would result in the members so treated.

Prestressed Concrete

Prestressed concrete has captured the imagination and various claims have been made regarding its economy over more traditional mediums. It does appear, however, that at the present stage of development, real advantage can only be gained by its employment when the self-weight of the structure it replaces is large in comparison with the superimposed loads to be carried. In addition, its peculiar characteristics in some cases render it more suitable for special jobs such as liquid retaining structures, or for tasks where the lower weight for handling or greater length of the clear span made possible, make a contribution to the economy of the particular structures.

In the early stages there may be a tendency by designers to use prestressed concrete on structures which cannot utilise its advantages to the full, and for contractors to cover themselves against contingencies which may arise from the use of a technique with which they are not fully familiar. This situation is a temporary one, and the stage is probably now being reached where wider knowledge mitigates against these objections.

As far as the construction itself is concerned, fears have been expressed as to its suitablity in resisting corrosion, fire and fatigue. Authorities appear to be agreed that these fears have no very substantial foundation.

The War Department considered the possibility of utilising prestressed or poststressed concrete for single-storey buildings, not with a view to saving cost, but primarily with the object of saving steel at a small cost disadvantage.

Initially it was proposed to use a reinforced concrete tied rafter construction, making a steel saving by the substitution of pretensioned prestressed concrete purlins for the normal reinforced concrete type. In a complete reinforced concrete design of this type, 24 per cent. of the total weight of the steel parts and reinforcement amounting to 1.42 lb. sq. ft. on an internal bay is contained in the purlins, but by the use of prestressed purlins a total inclusive steel weight 1.29 lb. sq. ft. of floor area was achieved, on an internal bay of a 48 ft. span building. The possibility of prestressing the rafters was also considered but abandoned owing to the expected high cost of anchorages on such short members.

A further design was prepared comprising reinforced concrete columns and post-tensioned prestressed gutter beams carrying self-supporting corrugated asbestos

arches. Thus the use of purlins was entirely eliminated in an attempt to compensate for the high cost of the prestressed beams and achieve near parity with orthodox construction. Details of this design are shown in Fig. 18.

the war. These war-time huts, however, were built at ground level and they were short in length so that some stiffening of the arch may have been derived from the close proximity of the gable ends. As a preliminary to the adoption of the construction, proving tests were

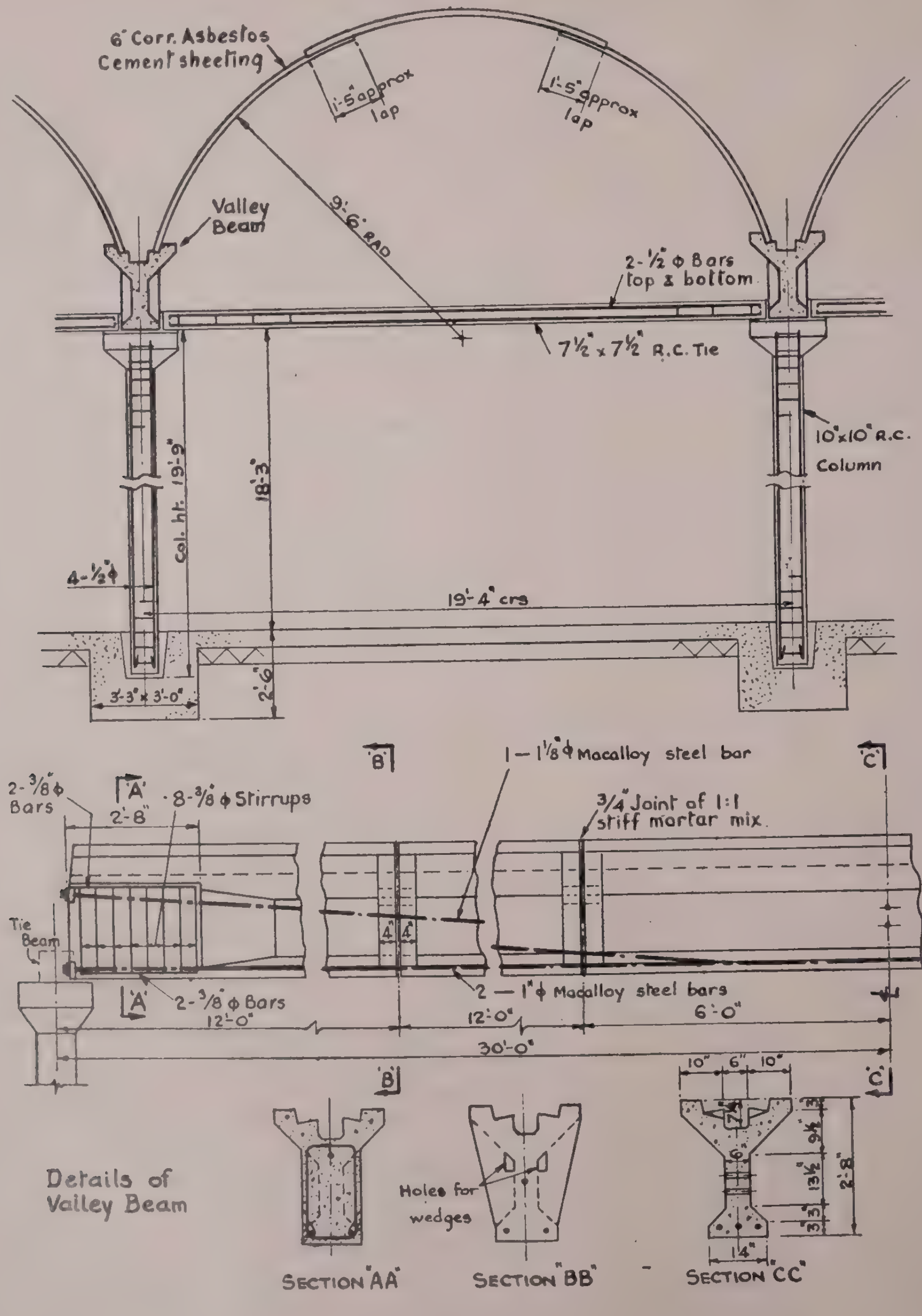


Fig. 18

It is considered that the employment of prestressed concrete in this manner is probably a misuse of the medium, although justified in this case by the low steel consumption, which amounts to 0.8 lb. sq. ft. of floor area for the whole structural frame calculated on an internal bay.

Although the use of 18 ft. span self-supporting asbestos arches of 6 in. corrugated curved sheets was considered a little ambitious, the construction is not entirely without precedent and huts on this principle were built during

carried out on Big Six and Handcraft sheets to assess the resisting moment of the curved sheeting under load. For convenience, the test load was applied at the crown of the arch and the resulting resisting moment calculated. A uniformly distributed load of approximately 2 cwt. per ft. run of arch was sustained at the crown in both cases before failure. A test rig. as shown in Fig. 19 was employed for the loading test.

The modulus of rupture of 1 ft. width of the sheets which were only one month old and not fully matured,

was calculated to be 3,150 lb./in., which was over twice the actual bending moment of 1,350 lb./in. imposed at any point due to the worst conditions of wind pressure, suction and superimposed loading under working conditions. (60. M.P.H. wind force ; 10 lb. sq. ft. superimposed load.) Initially load tests were carried out with the ends of the Big Six sheets locked by wedges to simulate fixed ends. These wedges were later released without any apparent loss in strength of the arch, although friction between the ends of the sheet and the concrete seating probably gave an end condition superior to pinned.

It should be noted, that, as far as pure strength considerations are concerned, the use of new sheets was probably an unduly severe test since asbestos cement goods will strengthen with age up to 10 years, although some slight embrittling will also take place. This latter feature might rule against the general adoption of the construction for arch spans approaching the size of that shown, until more is known of its behaviour over a number of years. In the case under consideration, a limited life only was required of the building, and a greater span than the asbestos cement manufacturers would normally recommend was therefore considered justified and adopted.

Fig. 19.—Big Six Asbestos Cement Sheeting under test

After a period of time, under fluctuating wind loading, it may transpire that the sheeting will tend to work at the laps and incur a higher maintenance than desirable. This is one of the lessons that remains to be learnt.

The roof as originally visualised included no natural lighting. This was later required and Perspex sheeting was introduced as the most suitable type, but its weakening effect was viewed with some slight misgiving. Perspex by itself has very adequate strength, but its "E" value is such that it would share little of the load as a skin stressed structure when fixed in combination with asbestos sheeting. Therefore, high local stresses along the edges of the asbestos sheets adjacent to each light will be present.

Acknowledgment is made of the tests carried out by the Universal Asbestos Man. Co., Ltd., and by Turners Asbestos Cement Co., Ltd., by whose courtesy the photograph Fig. 19 appears. This photograph shows the arch with the initial load of ½ cwt. per ft. run applied by means of a steel beam on the crown.

In designing in prestressed concrete, one factor for consideration is whether it is better to adopt pretensioned units or whether an advantage is to be derived from using post-tensioning. Pretensioned fully bonded units have the following advantages and disadvantages as compared with site post-stressed members.

The advantages are :—

(a) The high quality concrete can be rigorously controlled and more cheaply produced under factory conditions.

(b) No expensive end anchorages are required.

(c) The tensioning of the wires is more certain and guaranteed.

(d) Manufacture is accelerated.

(e) When using the smaller diameter wires, less steel can be used due to the higher stresses permitted.

The disadvantages are :—

(a) Unless factories are near the building site the transport may be costly and limitations in the sizes of members may be imposed.

(b) The long line system normally utilises wires which are straight from end to end of the bed so that a less economical section may have to be used for beams where the top flange tension at the beam ends is often a controlling factor.

(c) Handling stresses may in some cases dictate a stronger section.

(d) The economics of long line factory manufacture require work with fair repetition to occupy the casting beds.

As a broad principle it is felt that prefabrication should be adopted to keep labour off the site, especially where sites are remote from a source of skilled labour. However, post-stressing can be arranged so that by the introduction of dry mortar joints the units can be cast in a factory in convenient lengths for stressing at site, and thus retain many of the advantages of factory manufacture without some of the disadvantages.

The introduction of joints will not allow tension stress to be induced at the junction of the units but judicious positioning of them may minimise their weakening effect. This method was adopted in the design as the units were required to be cast upside down to ensure a sound casting round the difficult gutter profile. If a complete beam were cast in this manner special provisions for turning it over would have had to be made or, alternatively, it would require to be stressed in this position before handling, accepting some loss of the advantage of stressing against the self weight.

Other Considerations

Side Stanchions. In the design of cantilever side stanchions it would seem illogical that the maximum percentage of stress used in bending should be added to the percentage of the permissible axial stress based on an l/k reduction when these effects do not happen at the same point in the length of the shaft. The zone of column crippling is centred near the mid-height, whereas the maximum stress induced by wind moment occurs at the stanchion foot where the stanchion is acting as a short column as far as allowable axial compression is concerned. It is therefore suggested that the bending moment applicable to 1/5th height should be used instead of that at base when combining bending and axial column stresses.

In side stanchion arrangements it will in many cases be found advantageous to treat every other stanchion as a cantilever, alternating these with stanchions which

are supported horizontally at their caps by the truss diagonally braced back to the head of the cantilever stanchions. This arrangement is particularly suitable where valley beams are used—allowing the secondary side stanchions to be designed as simply supported beams with the wind forces conveyed back to the main bents.

As to sections, broad flange beams, double channels or single tubes create an economy when used as valley stanchions. Double tubes also have useful properties for eaves stanchions.

Walling. 5 in. thick dwarf concrete walls have been employed in War Office shed designs but have been found to be far more expensive than 9 in. concrete block walling, and both of course are dearer than asbestos sheeting on angle rails. Prototype sections of corrugated *in situ* walls in 12 ft. × 16 ft. panels comprising a total thickness of 1½ in. of gunned mortar on expanded metal are to be tried out but as yet little is known of the comparative cost of this construction.

Glazing. As has been shown earlier, glazing is a costly item, of which the flashings amount to an appreciable proportion of the total. Glazing normally takes the form of continuous runs of patent lead clothed or aluminium astragals with ¼ in. wired glass and will, erected, amount to about 5s. 9d. per sq. ft. for large areas when the costs of flashings, glazing purlin cills, and asbestos filler pieces to the roof sheeting at the head of the glazing are included.

There are various alternatives such as mastic bedded glass in steel astragals and also such types as Perspex or corrugated wired glass which may if required be used discontinuously on the pepper-pot principle, and which do away with the need for glazing cills and flashing. Corrugated glass which laps with corrugated sheeting is popular on the Continent and uses no lead and less steel than the conventional type : it is, however, restricted to spans of up to 5 ft. and at this span is possibly very slightly cheaper than the more substantial patent glazing when the saving on glazing cills, flashings and eaves fillers are taken into account.

There appears a real need for a device—possibly a cold formed purlin—which will combine the function of purlin, glazing cill and flashing to effect a glazing economy ; failing this, on narrow widths of glazing, corrugated glass offers some advantage although it gives perhaps a slightly lower standard than patent glazing.

Conclusions. In conclusion, it appears that some substantial reduction in the strength of purlins is possible ; further, that a suspended span purlin arrangement might well be employed. In addition, a more rational method of computing the safe stress in members subject to a combination of bending and direct stresses reaching their maxima at different points in the length of the member, is to be recommended.

The most economical grid for a building appears to lay in the neighbourhood of 50 ft. span and 12 ft. 6 in. spacing of trusses. Also the avoidance of valley beams will lead to an economy in the structure, but where this is not permissible the suspended span arrangement could be adopted with advantage.

Of the various types of frame, the traditional triangulated truss has much to commend it with the tied rafter frames comparable, and with both superior to the normal portal types.

Considering the various media of construction—trussed steel angle assemblies are adaptable for use on both large and small contracts with little fluctuation in price, and on this score, steel is the most generally acceptable medium. Where there is a fair repetition of similar units, welded tubular steel trusses in combination with angle or cold formed purlins will show an overall economy in weight, and in many cases in cost, over conventional riveted angle construction. For short spans, welded trusses fabricated from the lighter gauge cold formed sections, have an advantage in cost over all other types whilst giving an extremely favourable usage of steel.

Reinforced and prestressed concrete are also media which require a large measure of repetitive work to render them at all comparable to steel, but where this is assured they can achieve near parity on cost with angle trusses when the maintenance element is included, whilst making a very big contribution to steel economy. As compared with conventional angle construction, reinforced concrete uses about 40 per cent. of steel and prestressed concrete 23 per cent., whilst the latter requires less cement than the reinforced concrete structure. To get the maximum benefit from concrete construction it is necessary that precast units should be employed without excessive provision for handling stresses, the units being lifted the right way up at all stages after casting, by lifting devices.

Steel has one advantage over concrete in that it is sympathetic to plastic design and the use of two-pinned rafter frames designed on this basis with suspended span purlins may in the future prove competitive. Such a construction would allow site joints to be bolted to obviate the need for site welding which is always an expensive item.

Timber offers a reasonable, although perhaps a slightly more expensive alternative to the other constructions for spans up to 50 ft.—the limit for single connectors in the internal truss members. The price of timber trusses is however appreciably increased when a fire-resisting coating is required. Aluminium has not been considered within the range of trusses dealt with in this paper since the high cost of the material favours it only on exceptionally large spans where its low self-weight renders it appropriate ; or it may be favoured where its corrosion resisting properties are of value.

It is not claimed that this paper offers a complete solution nor that it covers all possible constructions. Such naturally insulated types of building as barrel vaults have been omitted since the sheds under consideration were to be to a semi-permanent standard and unheated so that the more expensive, but more substantial, constructions were excluded.

Some hesitation has been felt about entering the contentious field of costing but it is considered that the designer cannot carry out his task without a rough idea of the cost of the media which he employs and so an attempt has been made to indicate the probable trend. Experience with prices has shown, however, that unaccountable variations occur which render really accurate comparison on a cost basis a somewhat pious hope.

In conclusion, the author would like to acknowledge the assistance and helpful suggestions given by Mr. R. A. Huggins, B.Sc., during the preparation of this paper.

Discussion

The Literature Committee would be glad to consider the publication of correspondence in connection with the above paper. Communications on this subject intended for publication should be received by August 1st, 1954.

The Reconstruction of a Soaking Pit Building

Discussion on Paper by Mr. H. C. Husband and Mr. K. H. Best*

The CHAIRMAN (Lt.-Colonel R. F. Galbraith, M.C., B.Sc.(Eng.), M.I.Struct.E., A.M.I.C.E.), introduced the authors and expressed his great pleasure that one of the Institution's very dear Past Presidents, Professor J. Husband, was present on that occasion.

Mr. H. C. HUSBAND, on presenting the paper, thanked the Institution for having invited Mr. Best and himself to submit it for discussion. He also expressed his great pleasure in seeing his very old friend, Mr. George Mutch, of the Iron and Steel Division of the Ministry of Supply, present at the meeting ; he had been interested in the work described in the paper since its inception.

The PRESIDENT, proposing a very hearty vote of thanks to the authors for their paper, said it was a remarkable achievement to have obtained such graceful lines in structural steelwork as were indicated by the picture displayed at the meeting and by the last illustration shown on the screen. Unfortunately, many structural steel buildings were not graceful. The paper had been most interesting.

(The vote of thanks was accorded with acclamation.)

Mr. B. E. S. RANGER, A.M.I.C.E. (Associate-Member), offered his congratulations on a very interesting and most readable paper, and also on an exceedingly interesting set of progress photographs, which one had not the opportunity to see very often. One had not to stretch the imagination to read between the lines of the paper and to gain some idea of the complexity of the job, for it entailed erecting the structure in a most congested site, whilst at the same time production was not only maintained but, as the authors had said, actually increased.

The Vierendeel girders provided one of the many interesting features of the work. There were it appeared three cases to consider : (i) between the instrument room and the control room, (ii) the monitor roof, and (iii) at the back of the control room. Mr. Ranger was concerning himself with the reasons determining the adoption of that method of construction instead of a more conventional type using say, lattice girders.

The first case, between the instrument room and the control room, was an excellent example of fitness for purpose, where square openings for instrument panels were required. In the case of the monitors, possibly Vierendeel girders were necessitated by the type of ventilator required but it did seem that a suitable type of louvre could have been positioned outside the steelwork and a comparatively light welded lattice girder used.

The authors would have examined the economic aspects of the method of construction adopted, and he would be very interested to know their reasons for using Vierendeel girders for the monitors, and also for the girder at the back of the instrument room. There appeared to be only a door through in the latter case ; otherwise he could not see particular reasons for anything but a light lattice girder.

He was aware of the design requirements the authors had stated in para. 5 on page 305, that lattice girders and triangulated roof trusses were to be avoided to cut down the area for the lodgment of dirt and also to permit easy painting and maintenance ; but he thought that a lattice girder of say simple double angle box sections could be perfectly adequate in this respect.

While dealing with the Vierendeel girders, Mr. Ranger drew attention to the fact that the floor and roof beams of the control rooms were welded into the chords of the girders. That involved six welded joints and six site joints per bay, and he had been wondering whether the floor beams could not have run through either underneath or on top of the chords.

Another point which would interest many engineers was the deflection of the middle Vierendeel girder and he asked if the authors would be kind enough to indicate the total load and the actual measured deflection.

Turning to the main frames, there was a point which puzzled him a little in regard to the bases. Figs. 6 and 10 showed details of these ; the flanges were 21 in. × 1⅞ in., whereas it appeared that the whole load was carried in conjunction with the web plate by ¾ in. thick stiffeners. In similar cases, he had found that a more natural detail was obtained by bringing the flanges closer in on to a rocker bearing the full depth of the section, using a plate on either side parallel to the web to spread the load on the rocker.

In the site joints of the main frames, shown in Fig. 5, there was a separate tapered piece at the end of the flange plates which introduced an extra butt weld, and he believed its purpose was to avoid handling and machining the long flange. Bearing in mind that the flange had to be prepared for welding in any case, he wondered whether it was really a cheaper proposition.

In conclusion, Mr. Ranger said his remarks were not intended to detract from what was clearly a most efficient piece of planning and erection, the report of which he had read with considerable interest.

Mr. BEST thanked Mr. Ranger for his kind remarks. With regard to the reasons for adopting Vierendeel girders in the various locations, he said that perhaps it had not been made sufficiently clear that the rear girder in the control room was equally concerned with certain instruments. Each soaking pit required a separate group of instruments mounted in panels between the verticals of the front girder and there were in addition groups of switches and other electrical gear which were to be mounted at the rear of the control room. Most of this equipment required access from the rear and again it seemed logical to provide an open framed girder. Further, there was the question of the cantilevered

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*Read before a meeting of the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1, on Thursday, November 26th, 1953, Lt.-Colonel R. F. Galbraith, M.C., B.Sc. (Eng.), M.I.Struct.E., A.M.I.C.E., in the Chair. Published in THE STRUCTURAL ENGINEER, Vol. XXXI, No. 11, p. 301.

floor and roof the action of which demanded rigidity between the two vertical girders. Accordingly, each pair of verticals and each floor and roof member were designed as box frames. This construction is similar in some respects to the pair of open framed girders in the monitor roof in the main building, where the vertical posts, in addition to acting as members of the Vierendeel girders also provided legs of the portal frames forming the jack roof. This construction again provided the necessary lateral rigidity.

With regard to the connection of the floor and roof beams to the chords of the girders, it was very largely a question of space in addition to the question of transverse stiffness mentioned before. Probably it was not clear from Fig. 18 since the section here was deceptive. This illustration showed the portion of the existing Billet Mill roof which they were able to lift clear of the control room. At another section, this modification was not possible and clearances were very limited owing to the presence of an existing valley beam. At floor level, they were very severely limited owing to the height of the soaking pits and the space necessary to accommodate the lid mechanism and counterweight gear.

The design load for the central girder was approximately $11\frac{1}{2}$ tons at each panel point and the measured deflection under full working load was found to be approximately $\frac{5}{8}$ in., which compared well with the calculated figure.

The use of the additional tapered plate shown in Fig. 5 was requested by the fabricators of the steelwork, Messrs. United Steel Structural Co., Ltd., and agreed by the authors. As Mr. Ranger had pointed out, the long flange plate required to be prepared for welding in any case but the contractors preferred to handle the shorter length when machining the taper. With regard to whether this was really a cheaper proposition, there was no difference so far as their clients were concerned since the contractor's quotation was unaffected.

Mr. HUSBAND replied to Mr. Ranger's query concerning the base detail shown in Figs. 6 and 10. He agreed that flanges at the base of the frames appeared to be thick in relation to the base plate and stiffener detail. They had considered the possibilities of reducing the section and the flange thickness at the base but had decided against this for two principal reasons. In the first place, a big reduction in the depth of the section at this point did not give a good appearance, and secondly, they had in mind the fact that at floor level in this type of shop there was the distinct possibility that attachments to the main frames of the supports for service and plant may be required in the future. They therefore considered it wiser to provide an adequate thickness of metal in the flanges at these points.

Mr. H. N. SAYER, asked whether any special precautions were taken to prevent the heat from the soaking pits affecting the concrete in the column foundations.

Mr. HUSBAND said that the soaking foundations and flues were surrounded by considerable thicknesses of refractory bricks. In addition, the foundations for the main frames, and all other important foundations in the neighbourhood of the pits and flues, were protected by a layer of refractory concrete. Temperature measurements were taken to investigate the conditions and to ensure that the concrete was not "cooked." They had feared more trouble from this direction than had actually been experienced and had prepared a scheme for providing cooling water pipes in the foundations, but it was found that these were not necessary—the heat losse other directions had "saved their bacon."

Mr. R. H. SQUIRE (Member), who welcomed the pa very much, said that in order to appreciate it fully must have been more or less through the same sort mill as had the authors ; he had had 21 years of it various jobs. From the engineer's point of view completely new plant on a straightforward and cl site was often not specially interesting, but on a c gested site such as that on which the authors had to erect their building it was necessary to put in a lo thought and work, which added greatly to what mi be called the "fun of the game."

Referring to the fixings for the crane girders, he s Mr. Best had explained that there were machined c to fit the flange of the rails, and the holes were marl off and drilled on site. Mr. Squire made the po that on the soaker cranes there were very hea traversing crabs, and there was a correspondin heavy surge ; the spacing between the clips appeared be very wide. He asked whether any reliance placed on the friction of the rail on the crane girders whether the surge was entirely taken up by the rive and whether the rail itself was stiff enough to take surge between clips.

In connection with the ingot chariots, he said t from the drawing there appeared to be two tracks. was not quite sure about that, but inasmuch as ingot chariot was a very vital link between the soa pits and the mills, anything which interrupted service interrupted production completely. He did know how the chariots were propelled, but he had mind Messrs. Stewarts & Lloyds' plant at Corby, wh the track for the chariot came past the first two soak pits only ; it sometimes happened that the spuds slipp and an ingot dropped, and as this might occur over winch propelling the chariot, which was situated in soaker building, he had been asked to provide a co over the winch which would withstand the impact o 5-ton ingot dropped from a height of 8 feet. His fi impression was that, owing to the very limited sp available, this could not be done ; but by allowing fo considerable set in the main members, and balanc the work stored in the ingot against the work done producing this set, it was found possible to provide cover that would give protection, with the reservati that after one or two blows at maximum drop it wo be necessary to straighten or renew the cover. He ask whether the same problem had arisen in the soal building described in the paper.

Mr. HUSBAND, dealing with the last point, said t quite a lot of work in connection with protection fr falling ingots had to be done on the site. They h required to carry out some of the excavations a divert a very awkwardly placed culvert, which right across the site, while the old pits and chargers, a later the new chargers, were being used and protect cover had to be provided. Fortunately, in a large ste works there was usually a certain amount of steel ly about which could readily be used to construct adequa overhead cover. No very accurate calculations w necessary to devise these covers and one or two ing had in fact been dropped from time to time witho disaster. They had used what was virtually sc steel, i.e., which would normally have gone back to furnace, and had been able to provide very strong cov to protect the men and plant working below.

In reply to the first question, he said that they did allow friction between the rail tracks and the top flan

f the frame girder. The connections were designed to take the full horizontal surge of the chargers and were adequate at these centres. They had made many observations on the behaviour of the building and the crane girders since the new shop had been in use, and they had not found anything coming loose or adrift. The crane drivers liked the arrangement and felt it to be rigid.

A matter which had concerned the authors was that since all the stanchions or frame legs were identical these had the same natural frequency and it was considered that measures should be taken to ensure that oscillation caused by a nasty bump against the buffers would damp these out as quickly as possible. They had therefore put in heavy knee braces at the four corners of the building even though the stiffness of the connections and bracing was adequate in order to ensure that at least the four corner legs had a different frequency.

Mr. Best said the ingot chariots were not yet installed, but the present scheme was for them to be propelled by the usual steel wire rope haulage. The winches that would drive them would be provided with overhead cover to give protection against falling ingots, and in the final scheme the chargers would have rather less duties than at present. A bogie was extracted from the continuous furnace where the ingots were warmed, and it would be then transferred to the longitudinal track which was shown on the left-hand side of Fig. 20. One of the chargers would then lift the ingots therefrom and charge them into the soaking pits. The other charger would be used for drawing the hot ingots out of the pits and putting them on the right-hand chariot shown in Fig. 20. This would then be hauled by the steel wire rope haulage system to the north of the shop, where a cam was designed to throw over the projecting lug shown on the drawing and cant the chariot over, so that the ingot rolled down on to the mechanically propelled roller rack and was taken to the cogging mill.

Mr. D. E. Billington (Manager of the Works) added that the extra track was provided in order to allow for the operation of the two cranes ; the crane at the north end would be charging ingots at that end of the bay, and the other would be operating at the south end.

Dr. E. H. Bateman, (Member of Council), congratulated the authors on having used the Vierendeel trusses ; he considered that they contributed very much to the good appearance of the building.

He added that he was always interested in the adoption of this form of truss because he had produced a design analysis of it about 18 years ago.

Mr. J. A. Williams, A.M.I.C.E. (Member), remarked that before reading the paper he had been under the impression that he had been concerned with congested sites, but that with which the authors had been concerned was much worse than anything he had experienced.

Discussing the administration of the contract, he asked if any attempt had been made to invite competitive tenders, and if so, how much detail was given to tenderers. Was there the normal Specification and Bill of Quantities, or was the contract awarded, the main outline prepared, and then the work done on a more or less hand-to-mouth basis in conjunction with the Contractor ?

Mr. Husband replied that although there was of necessity some hand-to-mouth work, they prided themselves on having prepared a schedule of prices and approximate quantities and on having obtained tenders, so that the foundation work and a considerable amount of other work was on a competitive basis. With regard to the steelwork, there was an agreed price per ton for the supply and delivery of this to the site. The only major item of day-work was that concerned with the erection of the steel, and in this case they could not very well expect anyone to put in a firm price for that. The work of erection had to fit in completely with the operation of the works and production requirements took complete precedence over the building operations.

Mr. H. P. Holt (formerly of the United Steel Structural Co., Ltd), commenting on Mr Ranger's remarks, said that there seemed to be a slight misunderstanding regarding the webs and flanges of the main portals. The webs were centred. Although he agreed that it would look better to have the flanges brought in, the webs however had been checked and it was found to make no real difference.

Referring to the rail clips on the crane girder, he said they were site riveted. His previous company generally used machined clips and wedges, the former being welded in the shop on to the girder, the rail lined up on site, the wedges driven home and held in place by a loose bolt.

On the question of spacing of the clips, one speaker having asked whether they were too far apart, he agreed with the authors that the distance should be quite sufficient.

Mr. Husband said he was familiar with the type of clip referred to by Mr. Holt as being used by his company on their own works. The type actually used in the soaking pit building was one which the owners had adopted as standard at all their works and had found to be very successful. There was no sensible reason, therefore, why the authors should suggest an alternative—if things had gone wrong it would have been their fault.

About the rocker plates, he said that in the illustrations the arrangement looked perhaps a little awkward, but it was easy to fabricate. All the leg bottoms were in pits and were also surrounded with insulating and other flexible material to allow the very slight freedom that was necessary.

A speaker asked what was done about storing materials. There was a big yard, but there did not appear to be room inside.

Mr. Husband said that there were railway sidings not too far away from the site alongside which materials could be stored. There was by no means unlimited storage space but it was possible to stock a 100 tons or so between 100 and 200 yards away. The individual members were brought down to the site from these areas by steam crane as and when required for immediate erection, since there was not space at all on the site. It had been mentioned that the large open-framed girders were awkward to handle. These had been transported to the site by road and one was in fact slightly damaged when delivered but was straightened satisfactorily on site.

Mr. B. L. Clark (Associate-Member), congratulated the authors on a very readable paper and one which was interesting from all points of view.

About the action of the crane and the stability of the building, he asked if any effort was made to take the

inertia of the crane, should it get out of control and cause end shock, so that all the weight could be taken in the structure rather than relying entirely on the elasticity of the structure. In that connection he added that he was concerned, unfortunately, with a building of reinforced concrete in which there was a crane, and the owners were thinking of installing hydraulic buffers.

Mr. HUSBAND replied that the mass of the structure was of help in this direction and this was mainly concentrated at crane track level. Owing to the clear space required for plant below this level and the long spans of 60 to 70 feet, there were very heavy crane girders. There was also a second girder behind carrying roof beams across the span and there was a considerable amount of bracing in this area which helped. They had allowed in the design for a crane running at speed into the buffer stops, and the building was safe elastically. However, they were concerned that following a shock of this nature, the building might vibrate for some time, which, apart from being unpleasant, would not do the sheeting connections any good. The knee braces at the ends of the building had therefore been put in, not to prevent the building falling down, but in order to limit the time over which vibration might last.

The PRESIDENT, at the conclusion of the discussion, thanked the authors for the excellent way in which they had answered the questions.

Some Recent Foundation Research and its Application to Design

Written Discussion on Paper by Dr. G. G. Meyerhof (June, 1953) by R. H. Wood, Ph.D., A.M.I.C.E., A.M.I.Struct.E., A.M.I.Mech.E., (Building Research Station)

The author is to be congratulated on having tackled the difficult problem of linking the behaviour of the foundation with that of the structure, and with considerable success. It is obvious that a very wide field of research has been opened up, but it may not have been noticed by readers that this is one aspect of a more general approach to the problem of the behaviour of the complete structure, frame, encasement, floors, walls, foundations and all. In recent years very great advances have been made in the knowledge of the behaviour of bare steel frames, both under working (elastic) conditions, and also at "ultimate" or "plastic" collapse. Without in any way detracting from the merit of such work it is nevertheless clear from a study of Fig. 7 (c) of the paper that the stresses in the frame can *in addition* be modified, in some cases very considerably, by the other elements of the complete structure, most of all by floors and walls.

Fig. 1—Collapse of continuous beams (Stüssi)

Perhaps the best method of approach for an understanding of this problem is to refer to some comments made by Professor Stüssi in discussing the limitations of the "plastic" theories[1]. Stüssi pointed out that there was an inconsistency in "ultimate" theories which can be seen with reference to the collapse of the middle span of a continuous beam (Fig. 1) which carries a point load W, when the adjacent spans are unloaded and are of much greater length. Strictly speaking, the governing feature is the reduced *stiffness* of the adjacent members. The point at issue is that the plastic theories would demand an equalisation of moments ($\frac{Wl}{8}$ at centre and supports) whereas in the limit the support moment must become zero by virtue of the progressive reduction of stiffness of the adjacent members. In the limit therefore the solution is that of a simply supported beam($\frac{Wl}{4}$), and no abrupt transition is possible.

The present writer does not believe that this is a matter of any practical significance, *in an economically designed frame*, but that this "Stüssi Effect" is however of real practical significance in other cases, in the more general sense of composite structures. Thus there is a direct analogy between the continuous beam problem and the adjacent soil, failure of which can alter the collapse mode of the structure as a whole. This time we note that it is of considerable practical significance for Dr. Meyerhof has produced types of collapse modes of the models which have been hitherto unknown (for example the "Plastic hinge at leeward beam-column

Fig. 2—A distributed load of approximately 12 tons of cast iron weights on one of the floors at the new Government offices, Whitehall Gardens. The maximum stress recorded in the beams due to this load was only of the order of quarter-ton per square inch

joint only," page 163). In the opposite sense a general study of composite structure has already shown how the load can be transferred from a beam to adjacent walls which are this time of much greater stiffness[2]—the bending moment in the member being reduced to a value as low as $\frac{Wl}{300}$. Similarly, in the tests at Whitehall Gardens, to which the author has referred, very large reductions of stress took place in the frame due to the presence of floors, etc. Thus in Fig. 2 remarkably low stresses were recorded for a heavy floor load.

Fig. 3—The underside of a square floor supported by steel beams at failure. A plastic "fracture line" runs right across the centre of the slab and includes plastic hinges in the beams. A similar though less well developed system is at right angles to the former

The outcome of the author's investigations appears to be that a study of *deflections* is rapidly becoming an important subject. Thus it is necessary to guard against collapse, which is really unlimited deflection, whilst a study of settlement combined with an elastic analysis of the structure under working conditions appears to be necessary to limit the deflections at the working loads to prevent damage to partitions. In some cases the partitions themselves may be the deciding feature, notably in the case of brick walls.

The soil then is one part of the complete " composite " structure and similar alterations to the collapse modes of the supporting frame have been found by the writer. Thus Figs. 3 and 4 show photographs (at failure) looking up at the underside of a square reinforced concrete floor supported by beams on each side and carrying 16-point distributed load. In the first case the collapse mode produced " fracture lines " running right across the slab down each centre line, together with plastic hinges in each beam at the centre. In the second case (Fig. 4), because of some degree of asymmetry only the far beam has failed, the remaining three not taking part in the collapse mode at all, the fracture lines emanating from the near corners and the centre of the far beam—producing a curious Y pattern. Likewise, it is well-known that very stiff beams would produce a diagonal or X pattern of fracture in the slab. The important point here is that the supporting frame *may or may not take part in the collapse mode*, which is the same thing as saying that the loads which the frame receives are considerably modified and determined by the presence of the floors themselves.

This study of load distribution on the frame (interaction) is now receiving considerable attention.[3]

In conclusion, the writer would like to point out that the term " Differential settlement " appears to be used by the author with a different emphasis in Fig. 7 (b) from Fig. 7 (c). The former refers to a difference of time interval ; the latter draws attention to settlement at different positions.

Dr. G. G. MEYERHOF replies : The author is grateful to Dr. Wood for his kind remarks about the paper, and for providing additional evidence of the composite behaviour of structures. The encasement of frames, walls and floors have in the past generally been considered as dead load transmitted to the framework, and only a limited amount of composite action (e.g., tee and ell beams) was taken into account in estimating the stresses in and deformations of the framework. Similarly, secondary stresses due to shrinkage, temperature variation and differential settlement, when taken into account at all, were estimated on the bare structure.

Recent research has indicated the way to a more realistic approach in which a study is made of the complete composite structure under working and ultimate load conditions. In this way an attempt is made to take the stiffness of all structural elements as well as of the soil into account when estimating the stresses, deformations, stability and strength of the complete structure under load. Results of preliminary computations by Dr. Wood and the author during their collaboration on some of the above problems have indicated that this approach is likely to be more difficult to apply in practice under working (" elastic ") conditions than in the limiting (" plastic ") state of composite structures. Since both working and limiting conditions have to be investigated in a complete analysis, the greater effort required by designers in the future should be more than compensated by a more rational and economical design.

Fig. 4—A similar floor with stiffer beams. This time only the far beam has failed. A Y-pattern fracture line in the slab is consistent with the fact that the remaining three beams have not failed

References

[1]F. Stüssi. " Modern Trends in Steel Construction." THE STRUCTURAL ENGINEER, September, 1952.

[2]R. H. Wood. " Studies in Composite Construction. Part I : The Composite Action of Brick Panel Walls supported on Reinforced Concrete Beams." National Building Studies. Research Paper No. 13. H.M.S.O. London, 1952.

[3]R. H. Wood. " Studies of Composite Construction. Part II (In the press).

Institution Notices and Proceedings

ORDINARY GENERAL MEETING

An Ordinary General Meeting of the Institution of Structural Engineers was held at 11, Upper Belgrave Street, London, S.W.1, on Thursday, March 25th, 1954, at 5.55 p.m., Lt.-Colonel R. F. Galbraith, M.C., B.Sc., M.I.Struct.E., A.M.I.C.E. (President), in the Chair.

The following members were elected in accordance with the Bye-Laws. Will members kindly note that the elections, as tabulated below, should be referred to when consulting the Year Book for evidence of membership.

STUDENTS

BRADBURY, Timothy Ian, of Wolverhampton, Staffs.
FLETCHER, Peter Michael, of Preston, Lancs.
FORMAN, Donald, of Middlesbrough, Yorks.
REYNOLDS, Maurice Hugh, of Uttoxeter, Staffs.
WELFORD, Paul Alexander, of Chigwell, Essex.
WILLIAMS, Keith, of Bolton, Lancs.
WINDLE, Geoffrey Vincent, of Bolton, Lancs.

GRADUATES

ABELA, Edwin, B.Sc., B.E. & A. Malta, of Valetta, Malta, G.C.
BAILEY, Alan Frank, of Frome, Somerset.
BIRKETT, Douglas Raymond, of Middlesbrough, Yorks.
BOWERS, Peter Grierson, of Middlesbrough, Yorks.
BURNIKELL, Leonard John, of London.
CHAUDHURI, Debkumar, B.E.(Civil) Calcutta, of Calcutta India.
CLARKE, Michael Henry, of Eastbourne, Sussex.
EVANS, Derek Norman, of Smethwick, Staffs.
FRASER, Angus Stuart, B.Sc.(Eng.) Natal, of Bulawayo, Southern Rhodesia.
GHOSH, Asish Kumar, B.E.(Civil) Calcutta, of Calcutta, India.
GOODFELLOW, Reginald George, of Bromley, Kent.
HENDERSON, John Graham, of Billingham, Co. Durham.
HUGHES, Brian Devine, of Manchester.
JORDAN, Vincent Sidney, of London.
KOZLOWSKI, Alojzy, of London.
LAHIRI, Ramapada, B.E.(Civil) Calcutta, of Calcutta, India.
LAMBERT, Derek Myers, B.Sc.(Eng.) London, of Ruislip, Middlesex.
MENON, Rayirath Govinda, B.E.(Civil) Madras, of Poondi, Madras State, India.
NAIK, Pradhakar Vishwanath, B.E.(Civil) Bombay, of Bombay, India.
PRYLINSKI, Wlodzimierz Joseph, of London.
REDA, Kazimierz, of London.
RICKETTS, Alfred Martin, of London.
RODGER, William, of Greenock, Scotland.
WHITEHOUSE, Stanley Owen, B.Sc.(Eng.) London, of Dudley, Worcs.

MEMBER

SPARKES, Stanley Robert, Ph.D., M.Sc., D.I.C., A.M.I.C.E., of London.

TRANSFERS

Students to Graduates

BURNELL, Ian Derek, of Loughton, Essex.
MCCADDEN, Michael, of Salford, Lancs.

Graduates to Associate-Members

BUTTERFIELD, Roy, B.Sc.(Eng.) London, of Shipley, Yorks.
JARVIS, Anthony Peter, of Enfield, Middlesex.
LAMB, Allan Roy, of Ashtead, Surrey.
POOL, James Fraser, B.Sc.(Civil) Rand, of Coleshill, Birmingham.

Associate-Members to Members

BARRETT, John Catton, of Rawdon, nr. Leeds.
COLE, James Arthur, of Fremantle, Western Australia.
MACKETT, Norman Joseph Frederick, of Slough, Bucks.
ST. JOHN STOW, Eric Stanton, of Esher, Surrey.

Members to Retired Members

BUNNY, Matthew Henry John, of Shoreham-by-Sea, Sussex.
CHRISTMAS, Walter F., of London.

RE-ADMISSION

Associate-Member

MEASURES, Lionel Robert Emery, of Norwich, Norfolk.

OBITUARY

The Council regret to announce the deaths of ALFRED FRANCIS CORRIDON, JOHN ELLIS and GEORGE MCLEAN GIBSON (Members).

EXAMINATIONS, JULY, 1954

The Examinations of the Institution will next be held at centres in the United Kingdom and Overseas on July 13th and 14th, 1954 (Graduateship), and 15th and 16th (Associate-Membership).

EXAMINATION RESULTS, JANUARY, 1954

HOME CENTRES

The examinations were held at the usual centres in Great Britain in January, 1954. Sixty-three candidates entered for the Graduateship Examination, and 260 entered for the Associate-Membership Examination, making a total of 323. Of these, thirty-two candidates passed the Graduateship Examination, and forty-four candidates passed the Associate-Membership Examination. The names of the successful candidates are :—

GRADUATESHIP EXAMINATION

BARWIS, Edward Charles.
BILLINGTON, Roy.
BONTOFT, Anthony John.
BOOTH, Richard.
BOWLEY, Robert.
BROWN, Arthur Sydney.
BROWN, Francis Stanley.
CLAYDEN, Eric John.
COX, Bruce Albert.
DOWELL, David Keith.
FRANCIS, Rhys Hugh.
FRANKS, Stanley Fitzgerald.
HALL, Edward Tufnell.
HARTLE, George.
HOWARD, Kenneth Cecil.
HUNT, Douglas George.
HUNT, Henry William.
KLIMOWSKI, Stefan Jan.
MCHUGH, Patrick Thomas.
MACLACHLAN, Ian Hamilton.
MATTOCKS, Ronald.
MORRIS, Gordon Ronald.
ODEDAIRO, Ebenezer Olufunso.
PEACH, George Derrick.
PEARSON, Matthew Thomas.
SANVILLE, Stephen Colin.
SCHOFIELD, John Carl.
SHEPHERD, John Donald.
STEAN, John George.
SUMMERBELL, George Bernard.
SWANSON, Gordon.
VENIER, John.

ASSOCIATE-MEMBERSHIP EXAMINATION

ASTILL, Alan Walter.
BIRD, Brian Cecil.
BLOW, Leslie William Furse.
BRIDGE, Stuart Berry.
BROTTON, Derick Maxwell.
BURMAN, David Charles.
CHAPMAN, William Edwin.
CROWDEN, Brian Bertram.
DUNN, William Nicoll.
ELLIOTT, John Cameron.
GREEN, Reginald William.
GREENHALGH, Fred.
HARRISON, Richard Hayton.
HORTON, William John.
HUTTER, James Louis.
KACZKOWSKI, Tadeusz Adam.
KAFAROWSKI, Zygmunt.
KENNY, Alphonsus Jerome
KEY, David Edwin.
LAU FOO SUN.
LAW, Geoffrey Thomas.
LETMAN, John Albert.
LEVELL, Donald John.
MCKAY, Bernard John.
MALLICK, Prodyot Kumar.
MASON, John Francis.
NEEDHAM, Frederick Harold.
NEWBY, Frank.
PARSONS, Geoffrey Frank.
PATRICK, John George.
PERRY, Leonard Ernest Arthur.
PLANT, Gordon Vickers.
ROSE, Douglas Frederick.
SENIOR, Alan Gordon.
SMEDLEY, Alan.
SMITH, Robert Bernard Louis.
STONEBRIDGE, Marcus Allan.
STRINGER, Robert George Alexander.
TAYLOR, John Godfrey.
TYPROWICZ, Tadeusz Wladyslaw.
WALTON, Frank Thompson
WHITTAKER, Dennis Beatty.
WILLIAMS, Thomas Eifion Hopkins.
WOOLDRIDGE, Harold Albert.

FORTHCOMING MEETINGS

The following meetings will be held at 11, Upper Belgrave Street, London, S.W.1.

Thursday, May 27th, 1954

Ordinary General Meeting, for the election of members, 5.55 p.m. Annual General Meeting, 6 p.m.

Thursday, June 24th, 1954

Ordinary General Meeting for the election of members, 6 p.m.

SUMMER MEETING

The Summer Meeting of the Institution will be held at Birmingham, on Tuesday, May 18th to Friday, May 21st, 1954.

BENEVOLENT FUND

The Annual General Meeting of the Voting Contributors to the Institution of Structural Engineers' Benevolent Fund will be held at 11, Upper Belgrave Street, London, S.W.1, on Thursday, May 27th, 1954, at 6.30 p.m.

OVERSEAS REPRESENTATION

The Council have appointed Mr. W. M. Spence (Associate-Member) to be the Institution's Representative in Singapore in the place of Mr. K. D. Mathewson, who is returning to this country.

HAMMERSMITH SCHOOL OF BUILDING

The Council have agreed that the candidates who successfully complete the three-year course at Hammersmith School of Building (in Structural Engineering), and pass the internal examination, be granted exemption from the Graduateship Examination ; also that candidates who successfully complete the four-years' course and pass the internal examination be granted exemption from the "Theory of Structures (Advanced)" paper in the Associate-Membership Examination.

LONDON GRADUATES' AND STUDENTS' SECTION

A visit to a Soil Mechanics laboratory in London has been arranged for Saturday, May 15th, at 10 a.m. The number of visitors is strictly limited and those wishing to participate should make early application to the Hon. Secretary, J. A. Pope, 53, Cranleigh Drive, Leigh-on-Sea, Essex.

Hon. Secretary : J. F. S. Pryke, B.A.(Hons.), Bushcroft, Slipe Lane, Wormley, Herts.

BRANCH NOTICES

LANCASHIRE AND CHESHIRE BRANCH

Joint Hon. Secretaries : A. S. Sinclair, A.M.I.Struct.E., 17, The Circuit, Cheadle Hulme, Cheshire ; M. D. Woods, A.M.I.Struct.E., 58, Spring Gardens, Salford, 6.

MIDLAND COUNTIES BRANCH

Hon. Secretary : L. A. Firminger, A.M.I.Struct.E., 656, Chester Road, Erdington, Birmingham, 23.

GRADUATES' AND STUDENTS' SECTION

Hon. Secretary : H. L. Bramwell, 139, Wood Lane, Handsworth Wood, Birmingham, 20.

NORTHERN IRELAND BRANCH

A meeting of the Branch was held on Tuesday, January 19th, when Mr. T. J. S. Mallagh, M.A.I., M.I.C.E., gave an illustrated lecture on "Silos, with particular reference to a new method of prestressing." The lecturer spoke briefly of the advantages of silo storage over other forms such as lofts and bulkheads, and went on to describe a 2,000 ton silo completed recently to his design. In this building the bins were 11 in number, each 11 ft. 9 in. diameter by 74 ft. high, and were built of 18 in. × 9 in. × 4½ in. concrete blocks, round which twin-wire cables were wound at the required centres. The ends of these were anchored by concreting in at the junctions of the bins and the cables were then pulled together at staggered points by a special lever and held in the deflected or prestressed position by wire ties. The whole of the reinforcing was finally gunited, which had the effect of fixing it so that no further dependence was placed on the wire ties.

The interest shown in the description of the development and finalising of the method was evidenced by the number of questions at the end. The proceedings terminated with a vote of thanks to the lecturer, who stated that a further building of greater capacity was now in hand.

Hon. Secretary : A. H. K. Roberts, B.A., B.A.I., M.I.C.E.I., M.I.Struct.E., "Barbizon," 26, Dunlambert Park, Belfast.

SCOTTISH BRANCH

Hon. Secretary : G. Drysdale, A.M.I.Struct.E., "Niaroo," 33, Union Street, Motherwell, Lanarkshire.

SOUTH-WESTERN COUNTIES BRANCH

The Annual General Meeting of the Branch will be held at the Duke of Cornwall Hotel, Plymouth, on Friday, May 21st, 1954, at 7 p.m.

Joint Hon. Secretaries : E. W. Howells, M.I.Struct.E., 10-12, Market Street, Torquay, Devon ; C. J. Woodrow,

"Elstow," Hartley Park Villas, Tavistock Road, Plymouth, Devon.

NORTHERN COUNTIES BRANCH

Hon. Secretary: Captain O. Lithgow, A.M.I.Struct.E., 4, Stoneleigh Avenue, Acklam, Middlesbrough.

WALES AND MONMOUTHSHIRE BRANCH

The Annual General Meeting of the Branch will be held at the South Wales Institute of Engineers, Park Place, Cardiff, on Tuesday, May 11th, 1954, at 6.30 p.m.

Hon. Secretary : G. R. Brueton, A.M.I.C.E., A.M.I.Struct.E., 86, The Exchange, Mount Stuart Square, Cardiff.

WESTERN COUNTIES BRANCH

Hon. Secretary : E. Hughes, A.M.I.Struct.E., 23, Southdown Road, Westbury-on-Trym, Bristol, 9.

YORKSHIRE BRANCH

Hon. Secretary : E. Wrigley, A.M.I.Struct.E., 17, The Drive, Alwoodley, Leeds.

UNION OF SOUTH AFRICA BRANCH

Branch Hon. Secretary : A. E. Tait, B.Sc., A.M.I.C.E., A.M.I.Struct.E., P.O. Box 3306, Johannesburg. During week-days Mr. Tait can be contacted in the City Engineer's Department, Town Hall, Johannesburg. 'Phone : 34-1111, Ext. 257.

Natal Section Hon. Secretary : E. G. Bennett, A.M.I.Struct.E., c/o The Reinforcing Steel Co., Ltd., P.O. Box 49, Merebank, Durban.

Cape Section Hon. Secretary : R. Stubbs, M.I.Struct.E., The Reinforcing Steel Co., Ltd., P.O. Box 2692, Cape Town.

Book Reviews

Prestressed Concrete, by Dr. Kurt Billig. (London : Macmillan, 1952.) x + 470 pp. ; $8\frac{1}{2}$ in. $\times$ $5\frac{1}{2}$ in., 36s.

The appearance of this comprehensive review of development in prestressed concrete is to be welcomed by the student and practising structural engineer alike.

The science of design in this comparatively new medium is no longer the exclusive prerogative of the privileged few, but is rapidly becoming the concern of all structural engineers who cannot ignore the insistent claims justifiably made on behalf of prestressed concrete which has made such rapid progress in theory, research and construction, and which has given such performances as equal those of ordinary reinforced concrete and structural steel in a number of specialised fields.

Professor Billig's book provides a comprehensive survey of existing practice and constitutes a valuable handbook on the subject, giving in concentrated form most of the information available on this new constructional medium.

The first part deals with history and development, general data and fundamentals, specification of materials, production processes, plant required and typical examples. The second sets out in a clearly understandable manner the basic principles of design ; followed in the third section by actual design problems incorporating worked examples.

The author's courage in setting down a draft code of practice, despite the mixed reception his first proposals in this connection received, stamps him as a pioneer in this field—a fact which cannot be denied. The bibliography of references is comprehensive and conveniently classified to accord with the subject-matter of each chapter, but the formidable list of British and foreign patents included in the book leaves the ordinary practitioner in doubt as to whether he can design in prestressed concrete without infringement.

C. W. G.

Concrete Mix Design, by L. Boyd Mercer. (Melbourne Technical College Research Bulletin No. 2, 1953.) 59 pp., $12\frac{3}{4}$ in. $\times$ 8 in.

A practical method for the production of quality concrete is given in this bulletin, the application of the results of preliminary investigations at the Melbourne Technical College with the research programme at the University of Tasmania as presented in Research Bulletin No. 1 ("The Law of Grading for Concrete Aggregate," reviewed in THE STRUCTURAL ENGINEER, Vol. XXX, No. 4, April, 1952, p. 99). A Supplement is included which describes an application of the recommended method to a specific example.

Simplified Design of Roof Trusses for Architects and Builders, 2nd Edition, by Harry Parker. (New York : Wiley, 1953 ; London : Chapman & Hall.) 278 plus xiv pp. $7\frac{3}{4}$ in. $\times$ 5 in., 32s.

This book, first published in 1941 to cover a first course of training for those who have no previous knowledge of the design of roof trusses, includes graphical analyses and tables of coefficients of stresses, and is illustrated by numerous worked examples and problems for solution by the student.

Since the first edition, there have been changes in specifications and in the second edition certain allowable unit stresses have been increased and new column formulæ advanced to conform with these changes. A completely new section has been added dealing with timber connectors and their use in the design of roof trusses, and new tables and charts and new formulæ have been included to cover this section and the other changes in the book.

Iron and Steel Directory, 7th Edition. (London : The Louis Cassier Co., Ltd., 1953.) 386 pp., $8\frac{1}{2}$ in. $\times$ $5\frac{1}{2}$ in., 25s.

The Iron and Steel Directory, last published in 1950, has been brought up to date, the sections on the analysis of pig iron and the British Standard Specifications for ferrous materials having been completely revised, and a new table comparing British and American steel specifications included. The book is divided into five sections the first two listing pig-iron manufacturers, blast furnaces and iron founders, steelworks, steelfounders and makers. Section III gives a list of British Iron and Steel Groups, Associations and Societies and Scientific and Technical Institutions. A considerable amount of technical data and information is given in Section IV and the last section of the book contains a directory for buyers, together with addresses.

A Simplified Method of Design for Cylindrical Shell Roofs

By H. Tottenham, M.A.(Cantab.)

Synopsis

In the field of structural engineering the last two decades have been marked by a rapid expansion in the use of shell concrete. Although the analysis of this type of structure has been subject to many theoretical investigations the actual design method has advanced little. Most of the developments in the design methods have been in adding refinements to the mathematical theory of elasticity of shells ; the calculations for the design of barrel vault roofs have remained tedious and arithmetically cumbersome.

In this paper a simplified design method is presented. Tables of influence coefficients enable the whole of the calculations to be completed by means of a slide rule. A numerical example is given in detail. An attempt is made to interpret the results of these calculations with relation to the material of the shell—reinforced concrete, and in particular to the disposition of the reinforcement.

The redistribution of the direct, shear, and bending forces caused by changing the material from one possessing "homogeneous isotropic elastic" properties to one possessing those normally accepted for reinforced concrete is discussed in a qualitative manner.

On the basis of this discussion the author suggests what, in his opinion, is the information required from the design calculations, and concludes that the method outlined, whilst eliminating much of the cumbersome arithmetic, nevertheless gives ample information for the design of barrel vault roofs.

Introduction

The first cylindrical shell roofs were designed as arches spanning between two edge beams[1], and these latter carried the load back to the columns. It was observed that the deflections of these edge beams was by no means as great as was anticipated, and in the following designs the edge zone of the shell was taken as part of the edge beams, the remainder of the shell acting as an arch. The portion of the shell assumed to form part of the edge beam was gradually increased until recently the entire shell was incorporated.[2]

This simplified assumption for the behaviour of the shell is quite adequate provided that it has no deep edge beams and it is fairly long compared with its radius, and for the edge zone of short shells. In general, however, if used as a design method it would lead to rather thick shells as very large bending moments are indicated. This led to the development of the membrane method of design.

In the membrane state all the loads on the shell are transmitted by direct and shear forces in the plane of the shell. This method of design required certain boundary conditions such as vertical tangents at the edge of the shell. Therefore semi-circular and semi-elliptical cross-sections were used.

As concrete was difficult to place on the steep sloping sides of the shell, and because the edge and valley beams appeared to be excessively deep, attempts were made to design shells, the cross-section of which was a segment of a circle with vertical rectangular edge beams. The first successful attempt to be published was that by Finsterwalder[3] in 1933. He used Love's investigations into the mathematical theory of elasticity of shells, and separated the design problem into two parts.

1. The solution of the membrane condition of the shell under the section of the surface loads ; and
2. The solution of the "edge load problem" to determine the interaction between the shell and the edge beam.

The method he evolved has not sensibly been altered to date, the difference between the various design methods in vogue to-day lies in the choice of the fundamental equation of compatibility of the edge load problem.

The first compatability equation produced by Finsterwalder* is :

$$\left\{ \triangle^4 \left(\frac{\delta^2}{\delta y^2} + \frac{1}{R^2} \right)^2 - \frac{1}{R^2} \left(\frac{\delta^2}{\delta y^2} + \frac{1}{R^2} \right) \frac{\delta^4}{\delta x^4} \right\} w + \frac{12}{d^2 R^2} \frac{\delta^4 w}{\delta x^4} = 0 \quad . \ . \ . \ . \ . \ (a)$$

Where : R = radius of shell.
d = thickness of shell.
w = radial displacement of shell.
x is measured in longitudinal direction.
y is measured in circumferential direction.

This equation was corrected and made more complex by Dischinger[4] in 1935. By making a large number of approximations Schorer[5] produced in 1935

$$\frac{\delta^8 w}{\delta y^8} + \frac{12}{d^2 R^2} \frac{\delta^4 w}{\delta x^4} = 0 \quad . \ . \ . \ . \ . \ . \ (b)$$

For this he was severely criticised and many attempts at more exact solutions were made, e.g., by Jakobsen.[6] In 1947 Jenkins[7] produced a more compact "exact" equation :

$$\triangle^8 w + \frac{12}{d^2 R^2} \frac{\delta^4 w}{\delta x^4} = 0 \quad . \ . \ . \ . \ . \ . \ . \ . \ (c)$$

Of these equations, those due to Schorer and Jenkins are capable of direct analytical solution.

The basic differential equation for the edge load problem is only soluble by considering the edge load in the form of a Fourier series. Thus a constant edge load of unity is represented by the series :

$$1 = \frac{4}{\pi} \left\{ \cos \frac{\pi x}{L} - \frac{1}{3} \cos \frac{3\pi x}{L} + \frac{1}{5} \cos \frac{5\pi x}{L} . \ . \right\}$$

*The equations have been slightly altered in form to give a comparison.

The origin in the longitudinal direction is taken at the centre line of the shell, the end stiffening beams being at $x = \pm L/2$.

Fortunately the effect of the first term is predominant and this is the only one considered here. For this reason, in the following detailed description of the design methods a unit edge load $X = 1$ is represented by

$$X = \frac{4}{\pi} \cos \frac{\pi x}{L} \text{ and at } x = 0,\ X = \frac{4}{\pi}$$

From (b) and (c) we can get the two auxiliary equations :

$$m^8 + \beta^4 = 0$$
$$(m^2 - \gamma^2)^4 + \beta^4 = 0$$

respectively, where β and γ are characteristics of the shell dimensions.

The simplest method of estimating the accuracy of Schorer's approximation is to compare the roots of these two equations, this gives us the condition that the parameter γ should be small compared with unity.

It has been found, however, by comparing the results of the computations for a wide range of shells that this is not a true criterion and that for all shells with a value of γ less than .35 (which is the greatest value met by the author and thus tested) the Schorer approximation is adequate. Since Schorer's equation involves only one parameter of the shell dimensions the solution is very amenable for tabulation ; the tables presented in this paper are based on the Schorer approximation together with the assumptions made below.

At each edge of the shell there are basically four unknown quantities, either four deflections, or four forces, or any combination of these. It has been found in practice that the assumption that the edge of the shell has no rotation (i.e., has no change of slope) does not alter appreciably the forces in the shell. In the method presented here it is therefore assumed that there is no edge rotation, thus reducing the number of unknown quantities to three.

A further assumption is that the shell is symmetrical in form and has symmetrical loading about a longitudinal axis. This is frequently not the case but, as the effect of the edge forces dies out across the shell, we can for practical purposes consider any half shell as half of a symmetrical shell. Thus a structure as in Fig. (1a) would be considered in three parts :

(i) The left external half as half of a barrel as in (1b) ;
(ii) All internal halves as part of an infinite series of barrels as in (1c) ; and
(iii) The right external half as half of a barrel as in (1d).

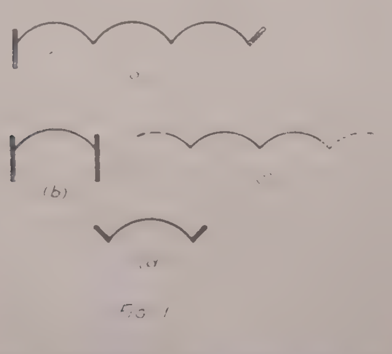


Fig. 1

The tables presented here are for symmetrical shells and cover a range of shells with a value of a characteristic parameter $K_1\ \Phi_k$ (see Appendix II) from 2.0 to 3.0. These cover the majority of shells met with in practice. A histogram (Fig. 2) covering 200 different shells dealt with at one design office in terms of number of occurrences of various values of $K_1\ \Phi_k$ in stages of

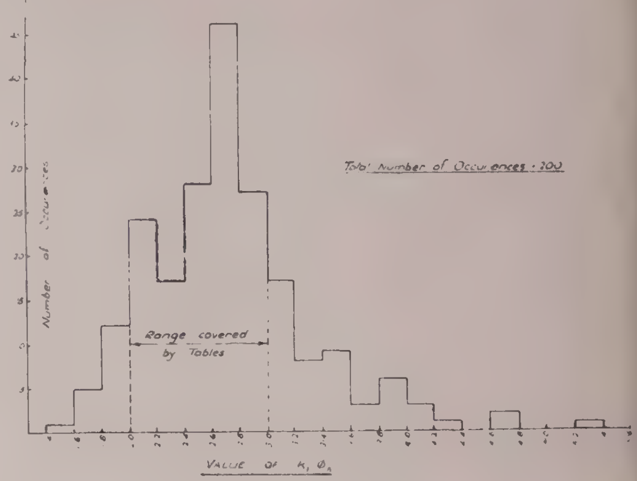


Fig. 2

0.2 shows the proportion that these tables cover. It is intended to extend the range of these tables to cover all the shells likely to be encountered, and also to publish tables for assymmetrical barrels such as " north light " and " butterfly " roofs.

It is useful to have a visual picture of the behaviour of a cylindrical shell. A typical barrel vault is shown in Fig. 3. Although the vaults have the appearance of

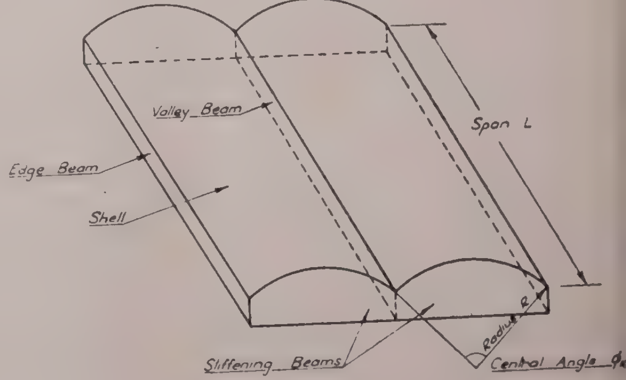


Fig. 3

arches, the load is essentially transferred to the end stiffening beams by the shell directly, whereas with arches it would be transferred via the edge and valley beams.

The most simple visual approach to the action of the shell is

(a) To picture the whole structure as a beam of open cross-section spanning between end stiffening beams and

(b) To picture the edge beams to be removed and the shell as a heavy piece of cloth fixed to rigid end stiffening beams. Under these conditions the edges of the cloth will sag downwards and inwards. The edge beams are then replaced, forcing the shell edge back to its original position.

The notation used in this paper is generally that used in the earlier works in shells, and each symbol is defined

either verbally or diagrammatically as it is introduced. A summary of notation is given in Appendix I.

The derivation of most of the expressions, and of the general theory of bending of shells, is incorporated in Appendix II. Further treatment of this aspect of the subject will be found in the references given at the end of the paper.

General Theory of Design Method

The general method of design is similar to that normally employed in the design of statically indeterminate structures. The sequence of operation is thus :

1. The structure is rendered statically determinate. This is accomplished by severing the edge beam from the shell. Provided that certain edge reactions are applied the forces and deflections in the shells are obtainable. This condition is termed the " membrane condition."

Under its own weight, any superload and the membrane reactions from the shell, the edge beam will also have certain deflections and stresses. (For convenience the displacements and the longitudinal stress at the edge of the shell will be collectively referred to as " edge displacements " ; and similarly the displacements and stress of the line of contact with the shell on the edge beam will be referred to as " beam displacements.") Thus there is a " gap " between the shell and the edge beam.

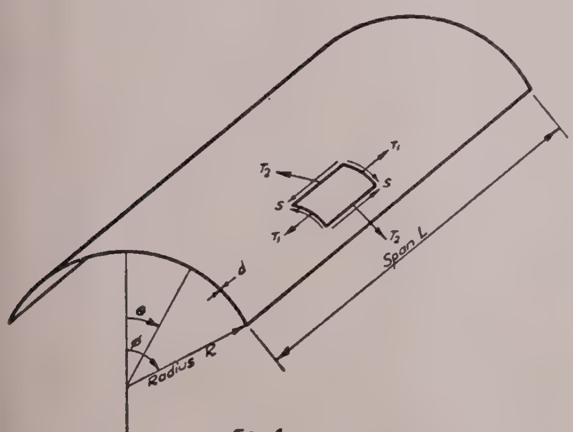


Fig 4

2. The influence of unit edge loads on the shell edge displacements are determined, as also are the beam displacements under similar unit loads.

From these the forces required to close the " gap " (i.e., to equalise the shell edge and edge beam displacements) are found.

3. The indeterminate forces in the shell due to these edge loads are determined at various points as required.

4. These indeterminate forces are added to the determinate (membrane) forces to give the complete stress distribution throughout the shell.

A. MEMBRANE FORCES AND DISPLACEMENTS

Assumptions

Apart from the usual assumptions of the mathematical theory of elasticity the following further assumptions are made :

(i) The shell thickness is small compared with the radius.

(ii) The shell does not carry any bending or twisting moments.

With these conditions the load is carried by direct forces, T_1 and T_2, and shear force S in the plane of the shell (Fig. 4).

Since there are three forces and three conditions of equilibrium of the shell (i.e., resolution of the forces in the normal, tangential and longitudinal direction), these forces represent a statically determinate system. The derivation of these forces is given in Appendix II. It will be noticed that the forces T_2 and T_1 are symmetrically disposed about the centre line of the shell in the x direction whereas S is antisymmetrically disposed. For this reason we use the rate of change of shear force dS/dx, which is symmetrical, in preference to S.

We have, then, at a point on the centre line along the shell, and at an angular distance θ from the crown :

$$T_2 = -\frac{4}{\pi} g R \cos\theta$$

$$\frac{dS}{dx} = +\frac{4}{\pi} 2g \sin\theta \quad \ldots \ldots \ldots 2$$

$$\text{and} \quad T_1 = -\frac{4}{\pi} \frac{L^2}{\pi^2} \frac{2g}{R} \cos\theta \quad \ldots \ldots \ldots 3$$

where: g = load/unit area of a vertical load uniformly distributed over shell surface.
R = radius of shell.
L = length of shell.

Also we get the vertical and horizontal deflections

$$\delta m_A = \frac{4g}{\pi d}\left(a \cos^2\theta + 4 + \frac{2}{a}\right) \quad \ldots \ldots \ldots 5$$

$$\delta m_B = \frac{4g}{\pi d} a \sin\theta \cos\theta \quad \ldots \ldots \ldots 6$$

where d = thickness of shell.

$$a = \left(\frac{\pi R}{L}\right)^2$$

In this paper (see Appendix I), all deflections have been multiplied by a factor of $\left(\frac{\pi^2 E}{L^2}\right)$ also the stresses in the influence coefficients are multiplied by -1.

With these factors the Clerk Maxwell relations hold for the influence coefficients, i.e.,

" displacement " in direction x due to unit force in direction y = displacement in direction y due to unit force in direction x, or $\delta xy = \delta yx$.

where the term " displacement " includes the three quantities considered, i.e., two displacements and longitudinal stress.

If then the shell has a half angle of Φ we have :

EDGE DISPLACEMENTS

$$\delta m_A = \frac{4g}{\pi d}\left(a \cos^2\Phi + 4 + \frac{2}{a}\right) \quad \ldots \ldots \ldots \mathrm{I}$$

$$\delta m_B = \frac{4g}{\pi d} a \cos\Phi \sin\Phi \quad \ldots \ldots \ldots \mathrm{II}$$

$$8^m{}_C = T_1/d = -\frac{4L^2}{\pi^2}\frac{2g}{Rd}\cos\Phi \quad \ldots \quad \text{III}$$

MEMBRANE REACTIONS

Vertical Reaction : $V^m = -T_2 \sin\Phi$
$$= +\, gR\cos\Phi\sin\Phi \quad \ldots \quad \text{IV}$$

Horizontal Reaction : $H^m = -T_2\cos\Phi = gR\cos^2\Phi \quad$ V

Shear : $$\frac{dS^m}{dx} = 2g\sin\Phi \quad \ldots \quad \text{VI}$$

B. SHELL EDGE DISPLACEMENT COEFFICIENTS

Assumptions

(i) Usual assumptions for the mathematical theory of elasticity.

(ii) The thickness of shell is small compared with radius of shell and length of shell between stiffeners.

(iii) There is no rotation of the shell at the edges.

(iv) Forces other than those shown in Fig. 5 are negligible.

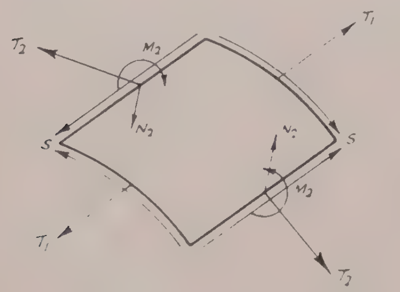


FIG 5

Notes on Assumptions

The first assumption is, of course, not true for reinforced concrete ; the effect of this will be discussed later. The second condition is more easily fulfilled. The ratios d/R and d/L are usually between 1/60 and 1/300, and 1/60 and 1/600 respectively.

The third assumption, as was mentioned in the introduction, is not strictly true except for the central internal edge of a symmetrical arrangement of barrels ; fortunately experience has shown that even in the case

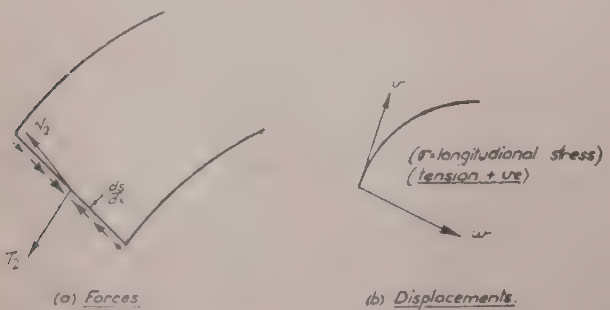


Fig. 6

of shells with free edges, such as north light construction, there is only small loss of accuracy in the determination of shell forces. The greatest inaccuracy is in the bending moments M_2, and since these are the most affected by temperature variations and other factors, they will be discussed later.

Application of Coefficients

In order to determine the displacement coefficients it is necessary to consider three types of edge loading on the shell. The forces applied to the shell are :

(i) tangential forces : T_2

(ii) radial force : N_2

(iii) shear force : dS/dx

and the three displacements are :

(i) tangential displacement : v

(ii) radial displacement : w

(iii) longitudinal stress : $T_1/d = \sigma$

The directions of the forces and displacements are as shown in Fig. (6).

The unit edge loadings are then Fig. (7)

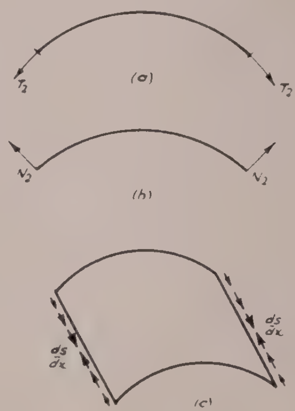


FIG 7

(a) Unit T_2, $\left(\text{i.e., } T_2 = \frac{4}{\pi},\ N_2 = \frac{ds}{dx} = 0\right)$

(b) Unit N_2, $\left(\text{i.e., } N_2 = \frac{4}{\pi},\ T_2 = \frac{ds}{dx} = 0\right)$

(c) Unit $\frac{ds}{dx}$, $\left(\text{i.e., } \frac{dS}{dx} = \frac{4}{\pi},\ T_2 = N_2 = 0\right)$

Using suffix notation we denote the value of the displacements due to load case (a) by v_T, w_T and σ_T, etc.

The Clerk Maxwell reciprocal relations give :

$$v_N = w_T, \quad v_S = \sigma_T, \quad w_S = \sigma_N$$

There are thus six coefficients required and these are given in Table I, where the factor $K_1\ \Phi_k$ is a shell parameter defined in Appendix II. The number given must be multiplied by the factor Q at the head of the column, i.e.,

$$v_S = \frac{R}{d^2\sqrt{r}} \times \ldots \left(r = \frac{\pi R}{L}\sqrt{\frac{\sqrt{3}R}{d}}\right)$$

To obtain the deflections coefficients in terms of the edge beam coordinates (i.e., vertical, horizontal, and longitudinal forces and deflections) we have : (Fig. 8)

FORCES $V = T_2 \sin \Phi - N_2 \cos \Phi$ 7

$$H = -T_2 \cos \Phi - N_2 \sin \Phi \quad . \quad . \quad . \quad . \quad 8$$

$$\frac{dS}{dx} = \frac{dS}{dx}$$

TABLE I.

$K_1 \phi_K$	v_T	$v_V = w_T$	$v_S = \sigma_T$	w_N	$w_S = \sigma_N$	σ_S
Q	$\frac{1}{d^3\sqrt{r}}$	$\frac{1}{d^3}$	$\frac{R}{d^2 r}$	$\frac{\sqrt{r}}{d^3}$	$\frac{R}{d^3\sqrt{r}}$	$\frac{R^2}{d^3 r\sqrt{r}}$
2 0	-11 30	+7 07	+7 34	-6 14	-3 81	-6 14
2 1	10 54	6 89	6 86	6 35	3 57	5 91
2 2	10 10	6 95	6 51	6 71	3 43	5 71
2 3	9 93	7 19	6 27	7 17	3 39	5 55
2 4	10 00	7·59	6 13	7 70	3 43	5 43
2·5	10·27	8 10	6·10	8 28	3 54	5·36
2·6	10·71	8·69	6 14	8 87	3 71	5·32
2·7	11·27	9 32	6 27	9 43	3 92	5·31
2·8	11·91	9 95	6 46	9 95	4 16	5 34
2·9	12·59	10 55	6·69	10 39	4 40	5 40
3·0	13 25	11 08	6 94	10·73	4 65	5 48

Thus

$$v_V = v_T \sin \Phi - v_N \cos \Phi, \text{ etc.}$$

DEFLECTIONS $\delta_A = -v \sin \Phi + w \cos \Phi$ 9

$$\delta_B = +v \cos \Phi + w \sin \Phi \quad . \quad . \quad 10$$

Thus

$$\delta_{AV} = -v_V \sin \Phi + w_V \cos \Phi$$

We obtain thus the nine coefficients

δ_{AV}	δ_{AH}	δ_{AS}
δ_{BV}	δ_{BH}	δ_{BS}
δ_{CV}	δ_{CH}	δ_{CS}

N.B. $\delta_{AH} = \delta_{BV}, \quad \delta_{AS} = \delta_{CV}, \quad \delta_{BS} = \delta_{CH}$

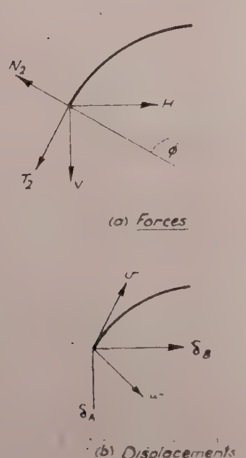


FIG 8

The dimensions of the displacement coefficients δ_A and δ_B are different from those of δ_C although certain of the coefficients are numerically equal. The longitudinal stress δ_C is in lbs./sq. ft., whereas the deflections δ_A and δ_B are multiplied by a factor $\frac{\pi^2 E}{L^2}$ and thus have dimensions lb./ft.[3].

C. EDGE BEAMS DISPLACEMENTS COEFFICIENTS

Assumptions

(i) The edge beam is elastic and can be considered as plain concrete.

(ii) The edge beam does not rotate about a longitudinal axis (i.e., is torsionally rigid).

(iii) The contact with the shell is along a horizontal line : for simplicity we take the springing line of the shell

Consider the beam shown in Fig. 9 having following dimensions :

Cross-sectional area $= A$

Length $= L$

M of I about NA_y and $NA_z = I_y$ and I_z respectively

Coordinates of line of springing $\bar{y}$ and $\bar{z}$.

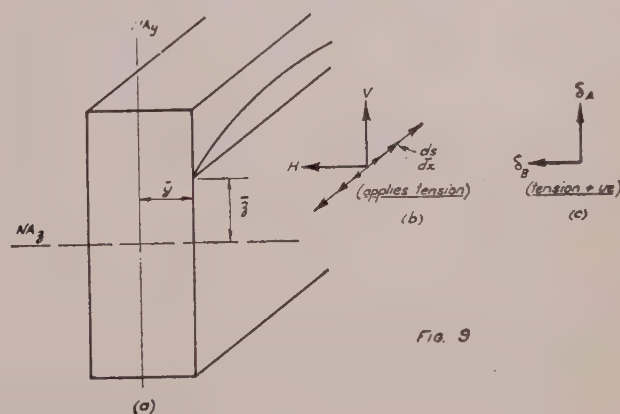


FIG. 9

We consider first action of line load $V = \frac{4}{\pi} \cos \frac{\pi x}{L}$ acting in direction shown.

$$\text{Then } BM. = \int_{-L/2}^{x} \int_{-L/2}^{x} -\frac{4}{\pi} \cos \frac{\pi x}{L}\, dx\, dx$$

$$= \frac{4L^2}{\pi\pi^2} \cos \frac{\pi x}{L}$$

$$\text{and deflection} = \int_{-L/2}^{x} \int_{-L/2}^{x} \frac{M}{EI}$$

$$= \frac{4}{\pi} \frac{L^4}{\pi^4} \frac{1}{EI_z} \cos \frac{\pi x}{L}$$

$$\text{Thus at } x = 0 \quad \delta_{AV} = \frac{4}{\pi} \frac{L^4}{\pi^4} \frac{1}{EI_z}$$

$$\therefore \text{ multiplying by } \frac{\pi^2 E}{L^2}, \; \delta^1_{AV} = \frac{4}{\pi} \frac{L^2}{\pi^2} \frac{1}{I_z} \quad . \text{ VII}$$

$$\text{Stress at springing, i.e., } z = \bar{z}, \; \delta^1_{CV} = \frac{4}{\pi} \frac{L^2}{\pi^2} \frac{\bar{z}}{Iz} \quad \text{VIII}$$

similarly, due to $H = \frac{4}{\pi} \cos \frac{\pi x}{L}$

we have $\delta^1_{BH} = \frac{4L^2}{\pi\pi^2} \frac{1}{Iy}$ and $\delta^1_{CH} = \frac{4L^2}{\pi\pi^2} \frac{\bar{y}}{Iy}$ IX, X

Also $\delta_{BV} = \delta_{AH} = 0$

considering line load $\frac{dS}{dx} = \frac{4}{\pi} \cos \frac{\pi x}{L}$

Integrating $S = \frac{4}{\pi} \frac{L}{\pi} \sin \frac{\pi x}{L}$

This puts a tensile force of $T = -\int_{-L/2}^{x} S dx$ on cross section $X = x$.

$\therefore T = \frac{4L^2}{\pi\pi^2} \cos \frac{\pi x}{L}$

Thus we have a section with an eccentric tension, there is a vertical bending moment $= T\bar{z}$ and a horizontal bending moment $= T\bar{y}$.

Thus vertical deflection $= \int_{-L/2}^{x} \int_{-L/2}^{x} \frac{-T\bar{z}}{EIz} dx\, dx$

$\therefore$ at $x = 0$ multiplying by $\frac{\pi^2 E}{L^2}$

$$\delta^1_{AS} = \frac{4L^2\bar{z}}{\pi\pi^2 Iz} \quad . \quad . \quad \text{XI}$$

Similarly $$\delta^1_{BS} = \frac{4L^2\bar{y}}{\pi\pi^2 Iy} \quad . \quad . \quad \text{XII}$$

The stress due to $\frac{ds}{dx}$ is $\frac{T}{A} + \frac{Tz^2}{Iz} + \frac{T\bar{y}^2}{Iy}$

$$\delta^1_{CS} = \frac{4L^2}{\pi\pi^2}\left(\frac{1}{A} + \frac{\bar{z}^2}{Iz} + \frac{\bar{y}^2}{Iy}\right) \quad . \quad . \quad . \quad . \quad . \quad \text{XIII}$$

Thus we have nine coefficients as with the shell.

D. MEMBRANE EDGE BEAMS DISPLACEMENTS

Under the membrane reactions and own load q we have the membrane edge beam displacements

$$\delta^1_{AM} = -(q+V^m)\,\delta^1_{AV} + \frac{dS^m}{dx}\,\delta^1_{AS} \quad . \quad . \text{XIV}$$

$$\delta^1_{BM} = H^m\,\delta^1_{BV} + \frac{dS^m}{dx}\,\delta^1_{BS} \quad . \quad \text{XV}$$

$$\delta^1_{CM} = -(q+V^m)\,\delta^1_{CV} + H^m\delta^1_{CH} + \frac{dS^m}{dx}\,\delta^1_{CS} \quad . \quad \text{XVI}$$

where V^m H^m and $\frac{dS^m}{dx}$ are given by equations IV to VI.

E. SETTING UP EQUATIONS FOR THE FORCES V, H AND $\frac{dS}{dx}$

Under membrane conditions the shell has displacements :	δ^m_A	δ^m_B	δ^m_C
Under membrane conditions the edge beam has displacements :	δ_{AM}	δ_{BM}	δ_{CM}
$\therefore$ there is a "*gap*" between shell edge beams of :	$\delta^m_A + \delta_{AM}$	$\delta^m_B + \delta_{BM}$	$\delta^m_C + \delta_{CM}$
These we denote by :	$\triangle_{AM}$	$\triangle_{BM}$	$\triangle_{CM}$
A force V applied to shell causes displacement :	δ_{AV}	δ_{BV}	δ_{CV}
A force applied to edge beams causes displacement :	δ^1_{AV}	δ^1_{BV}	δ^1_{CV}
$\therefore$ this force closes "gap" by :	$\delta_{AV} + \delta^1_{AV}$	$\delta_{BV} + \delta^1_{BV}$	$\delta_{CV} + \delta^1_{CV}$
We denote this by :	$\triangle_{AV}$	$\triangle_{BV}$	$\triangle_{CV}$
In a similar manner H closes gap by :	$\triangle_{AH}$	$\triangle_{BH}$	$\triangle_{CH}$
$\frac{dS}{dx}$ closes gap by :	$\triangle_{AS}$	$\triangle_{BS}$	$\triangle_{CS}$

Thus in the vertical direction we must have

$$V\triangle_{AV} + H\triangle_{AH} + \frac{dS}{dx}\triangle_{AS} + \triangle_{AM} = 0$$

and similarly

$$V\triangle_{BV} + H\triangle_{BH} + \frac{dS}{dx}\triangle_{BS} + \triangle_{BM} = 0$$

and

$$V\triangle_{CV} + H\triangle_{CH} + \frac{dS}{dx}\triangle_{CS} + \triangle_{CM} = 0$$

From these three equations we can find the forces V, H and $\frac{dS}{dx}$

F. DISTRIBUTION OF FORCES THROUGHOUT SHELL

The forces in the shell are :

(i) those due to membrane state

(ii) those due to edge loads $\left(V, H \text{ and } \frac{dS}{dx}\right)$

(indeterminate forces).

(i) MEMBRANE FORCES. These are given by equation (1) — (3)

(ii) Secondary Forces. To determine these forces it is first necessary to resolve the forces V and H into forces T_2 and N_2

i.e., $T_2 = V \sin \Phi - H \cos \Phi$
$N_2 = V \cos \Phi - H \sin \Phi$

The values of the five forces T_1 T_2 $\frac{dS}{dx}$ N_2 and M_2 due to unit edge load forces T_2 N_2 and $\frac{dS}{dx}$ are given in Tables IIa-IIe for the edge, 1/8, 1/4, 3/8 points and crown of the shell. In order to use these tables it is necessary first to reduce the edge forces by the appropriate factor Q. That is, if we denote the factor for T_2 by Q_{T_2}, etc., our reduced edge loads are T_2/Q_{T_2} N_2/Q_{N_2} $\frac{dS}{dx}/Q_S$ and the value of M_2 at the quarter point of the shell is then

$$M_2\ (1/4) = Q_{M_2}\left\{\frac{T_2}{Q_{T_2}} \times \ldots + \frac{N_2}{Q_{N_2}} \times \ldots + \frac{\frac{dS}{dx}}{Q_S} \times \ldots\right\}$$

the numbers being obtained from the tables.

The final force distribution is then the sum of the membrane and indeterminate forces.

The forces so obtained need a further correction. The loads we have applied have been the first term in the Fourier series for unity.

$$1 = \frac{4}{\pi}\cos\frac{\pi x}{L} - \frac{4}{\pi}\,\frac{1}{3}\cos\frac{3\pi x}{L} + \ldots$$

It is thus necessary to divide the results by $\frac{4}{\pi}$. For shear we have obtained $\frac{dS}{dx}$, in order to get the maximum shear it should be multiplied by $\frac{L}{2}$ giving the shear at the end stiffeners.

The value of T_1 obtained is also due to this loading term and represents the bending moment due to this load ; this is $\frac{4}{\pi}\,\frac{L^2}{\pi^2}$ whereas it should be $L^2/8$; it is necessary to multiply further by $\frac{\pi^2}{8}$

Checks on Calculations

In view of the quantity of arithmetic involved in the calculations it is advisable to apply numerical checks

T_1

TABLE II A

$Q_{T_1} = \frac{2\sqrt{3}}{d}$

	EDGE			1/8 POINT			1/4 POINT			3/8 POINT			1/2 POINT (CROWN)		
$K_1\Phi_K$	T_2	N_2	$\frac{ds}{dx}$	T_2	N_2	$\frac{ds}{dx}$	T_2	N_2	$\frac{ds}{dx}$	T_2	N_2	$\frac{ds}{dx}$	T_2	N_2	$\frac{ds}{dx}$
2.0	+2.118	-1.208	-2.736	+.630	-.262	-1.204	-.298	+.200	-.148	-.779	+.364	+.435	-.924	+.387	+.650
2.1	1.981	1.132	2.649	.550	.195	1.138	.292	.204	.132	.697	.300	.429	.811	.304	.607
2.2	1.879	1.089	2.561	.475	.132	1.075	.293	.214	.114	.622	.243	.404	.705	.218	.564
2.3	1.809	1.017	2.490	.402	.071	1.014	.301	.230	.094	.554	.192	.379	.604	.136	.521
2.4	1.770	1.089	2.438	.331	-.012	.954	.316	.252	.071	.489	.144	.355	.505	+.058	.475
2.5	1.760	1.124	2.402	.260	+.046	.893	.337	.278	.045	.429	.100	.329	.407	-.019	.427
2.6	1.774	1.177	2.385	.187	.104	.832	.365	.308	-.016	.372	.059	.304	.309	.094	.376
2.7	1.810	1.243	2.384	.115	.162	.771	.397	.341	+.017	.316	+.020	.277	.212	.167	.321
2.8	1.865	1.318	2.398	+.042	.218	.708	.434	.374	.052	.262	-.016	.251	.116	.237	.264
2.9	1.931	1.397	2.424	-.029	.271	.645	.472	.406	.090	.212	.050	.223	-.024	.302	.205
3.0	2.004	1.473	2.460	-.099	.321	.582	.511	.435	.129	.163	.081	.195	+.068	.360	.144

AMENDMENTS: Col. 11. For .262 read + .263 Col. 13. For + 435 read + .455 Col. 14. For — .024 read — .022

$\frac{ds}{dx}$

TABLE II B

$Q_S = \frac{1.554\,r\sqrt{r}}{R^2}$

	1/8 POINT			1/4 POINT			3/8 POINT		
$K_1\Phi_K$	T_2	N_2	$\frac{ds}{dx}$	T_2	N_2	$\frac{ds}{dx}$	T_2	N_2	$\frac{ds}{dx}$
2.0	+.936	-.487	-.097	+1.023	-.485	-.547	+.620	-.273	-.413
2.1	.900	.454	.100	.963	.429	.542	.574	.225	.407
2.2	.870	.428	.105	.905	.365	.536	.527	.177	.398
2.3	.846	.409	.111	.846	.309	.527	.478	.127	.386
2.4	.827	.398	.120	.788	.253	.516	.425	.075	.370
2.5	.815	.392	.130	.727	.198	.502	.368	-.020	.349
2.6	.809	.391	.143	.665	.142	.484	.305	+.037	.324
2.7	.808	.395	.158	.600	.085	.463	.238	.097	.294
2.8	.812	.403	.177	.533	-.029	.438	.166	.159	.259
2.9	.820	.413	.198	.464	+.027	.409	.091	.220	.218
3.0	.831	.424	.221	.395	+.081	.378	.014	.279	.174

AMENDMENTS: Col. 3. For .454 read .453. Col. 6. For .429 read .423. Col. 10. For .324 read .325

when possible. Such checks at the end of the calculations can be made by considering the equilibrium of the shell.

(i) The vertical components of the shear forces at the ends of the shell must balance the load on the shell. The vertical component of the shear is S sin and the total can be found by means of Simpson's rule. The vertical loads on the shell are :

(a) own weight + super load.
(b) membrane reaction.
(c) edge load V.

(ii) The total longitudinal compression in the shell must balance the tension imparted to the edge beam by the shear forces.

The total compression can be found by Simpson's rule and the total tension in the edge beam is given by $T = \dfrac{S\ L}{4}$

(iii) The bending moment on the shell is counterbalanced by the T_1 forces. If we take the moments about the springing line we have $M = \Sigma\, T_1 y$ when y is the rise to the point on the shell above springing line.

T_2 TABLE IIc $Q = \frac{r}{R}$

	1/8 POINT			1/4 POINT			3/8 POINT			1/2 POINT (CROWN)		
$k\phi_k$	T_2	N_2	$\frac{ds}{dx}$	T_2	N_2	$\frac{ds}{dx}$	T_2	N_2	$\frac{ds}{dx}$	T_2	N_2	$\frac{ds}{dx}$
2·0	+·799	+·255	−·424	−·084	+·695	−·096	−·809	+1·023	+·344	−1·061	+1·143	+·530
2·1	·791	·255	·442	·091	·670	·098	·802	·966	·358	1·066	1·067	·550
2·2	·780	·259	·458	·100	·650	·099	·792	·906	·371	1·045	·989	·567
2·3	·765	·268	·472	·112	·640	·098	·779	·851	·382	1·017	·909	·581
2·4	·745	·281	·483	·128	·633	·094	·761	·793	·390	·980	·825	·588
2·5	·721	·300	·491	·149	·633	·088	·740	·734	·396	·934	·735	·590
2·6	·690	·325	·495	·174	·638	·078	·712	·674	·397	·876	·639	·583
2·7	·653	·356	·494	·204	·648	·065	·679	·611	·395	·806	·537	·568
2·8	·609	·391	·488	·240	·663	·047	·639	·546	·388	·723	·429	·543
2·9	·559	·431	·478	·280	·682	·026	·595	·480	·376	·630	·316	·508
3·0	·504	·473	·462	·325	·704	·000	·546	·414	·360	·528	·202	·465

AMENDMENTS : Col. 6. For .650 read .652. Col. 13. For .508 read .509

N_2 TABLE IID $Q_{N_2} = \frac{1{\cdot}098\sqrt{r}}{R}$

	1/8 POINT			1/4 POINT			3/8 POINT					
$k_1\phi_k$	T_2	N_2	$\frac{ds}{dx}$	T_2	N_2	$\frac{ds}{dx}$	T_2	N_2	$\frac{ds}{dx}$			
2·0	−·551	+1·226	+·155	−·733	+·989	+·301	−·495	+·552	+·233			
2·1	·577	1·224	·170	·763	·980	·328	·513	·542	·255			
2·2	·602	1·220	·185	·790	·967	·356	·528	·530	·276			
2·3	·625	1·215	·201	·813	·949	·384	·539	·512	·295			
2·4	·647	1·209	·216	·830	·926	·409	·544	·489	·313			
2·5	·667	1·200	·231	·841	·897	·432	·543	·460	·327			
2·6	·685	1·190	·245	·844	·861	·452	·534	·424	·338			
2·7	·701	1·177	·259	·838	·817	·466	·516	·381	·344			
2·8	·714	1·162	·271	·822	·766	·476	·488	·329	·344			
2·9	·723	1·145	·282	·796	·707	·479	·451	·272	·337			
3·0	·730	1·127	·291	·761	·643	·475	·403	·208	·323			

M_2 TABLE IIE $Q_{M_2} = 1$

	EDGE			1/8 POINT			1/4 POINT			3/8 POINT			1/2 POINT (CROWN)		
$k\phi_k$	T_2	N_2	$\frac{ds}{dx}$	T_2	N_2	$\frac{ds}{dx}$	T_2	N_2	$\frac{ds}{dx}$	T_2	N_2	$\frac{ds}{dx}$	T_2	N_2	$\frac{ds}{dx}$
2·0	+·578	−1·306	−·197	+·399	−·544	−·161	−·011	+·135	−·015	−·400	+·609	+·158	−·556	·788	+·233
2·1	[illegible]	1·365	·226	·436	·566	·184	·013	·143	·017	·437	·634	·181	·607	·809	·267
2·2	·689	1·422	·257	·473	·586	·209	·015	·152	·018	·472	·657	·206	·657	·835	·303
2·3	·744	1·475	·290	·510	·602	·236	·019	·162	·020	·510	·676	·232	·705	·856	·340
2·4	[illegible]	[illegible]	·324	·534	·613	·263	·024	·172	·022	·544	·691	·258	·749	·869	·378
2·5	·849	1·565	·358	·574	·620	·289	·029	·185	·022	·575	·700	·283	·787	·870	·415
2·6	·895	1·601	·390	·600	·620	·314	·037	·198	·022	·601	·704	·308	·817	·869	·449
2·7	·935	1·628	·421	·619	·612	·337	·047	·214	·021	·621	·700	·330	·837	·852	·479
2·8	·967	1·646	[illegible]	·631	·597	·357	·060	·231	·019	·633	·686	·349	·844	·822	·503
2·9	·991	1·656	·472	·635	·573	·372	·075	·251	·014	·637	·668	·364	·836	·778	·519
3·0	1·006	1·657	·490	[illegible]	[illegible]	·382	·093	·272	·008	·632	·642	·374	·813	·722	·526

It would be possible to have further checks but the three above show that the structure satisfies the laws of statics in a vertical direction.

NUMERICAL EXAMPLE

The design calculations are given in full, just as they would appear in actual practice. As there are many coefficients, etc., which are required many times, a drawn-up sheet is advisable. Although a slide rule is used throughout it will be noticed that undue "accuracy" has been given in many figures. In the initial part of the calculations it is advisable to guess at an extra figure as this tends to keep down the errors which are normal with a slide rule (i.e., in line 17 we have 260.5, the last figure being approximate).

The shell has a span of 78 ft. between stiffening beams and other principal dimensions as in Fig. 10. The loading is taken as :

Self-wt. of shell ...	...	32 lb./sq. ft.
Live load ...	...	15 ,, ,,
Waterproofing ...	...	2 ,, ,,
		50 lb. sq./ft.

The design is carried out on Design Sheets I-IV.

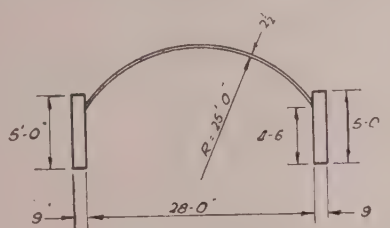


FIG 10

It is first necessary to calculate the shell parameters and other dimensions ; this is carried out in lines 1-5 (Sheet I). The derivation of the parameters is given in Appendix II.

The membrane reactions (lines 6-8) and displacements (lines 9-11) are worked out using formulæ I-VI and Section A.

The shell edge displacements are found in lines 12-17 using Table I. These are resolved into the edge beam co-ordinate system in lines 18-26, as shown in Section B.

Next, the edge beam displacement coefficients (lines 27-34) and membrane displacements (lines 35-37) (Design Sheet II) are evaluated using formulæ from sections C and D. From the above are set up equations for V, H and dS/dx (lines 38-46, see Section E).

As there are only three unknowns it is best, on a slide rule, to solve these by eliminating two of the variables to obtain the third. The first two are then found by substituting the found values as in lines 47-54. These redundant forces are then resolved into the directions T_2 and N_2 (lines 55-56, sheet III). The multipliers for the edge load forces in the shell and the "reduced forces" at the edge are worked out (lines 57-59). It is convenient to remove the term $\frac{4}{\pi}$ which arose from the use of the Fourier series at this stage.

The distribution of the forces in the shell are then calculated. The membrane system is worked out using the formulæ 1-3 (lines 60-65) and these values are added to the forces due to the edge loads which are obtained using Tables IIa-IIe (lines 66-97).

The checks shown in lines 98-107 show sufficient agreement for engineering calculations. The maximum error is only of the order of 1.5 per cent. These checks, however, only apply to the force distribution and do not show that the boundary conditions are satisfied. The boundary conditions can only be checked by an independent evaluation of the shell and edge beam displacements but the overall picture of the shell forces generally shows up any mistake.

Fig. 11 shows graphs of the force distributions in the shell. As a matter of interest a comparison of these results and those obtained by more accurate analysis is given in Appendix III.

REINFORCEMENT

Having found the theoretical forces in the shell the quantity and disposition of the reinforcement must be

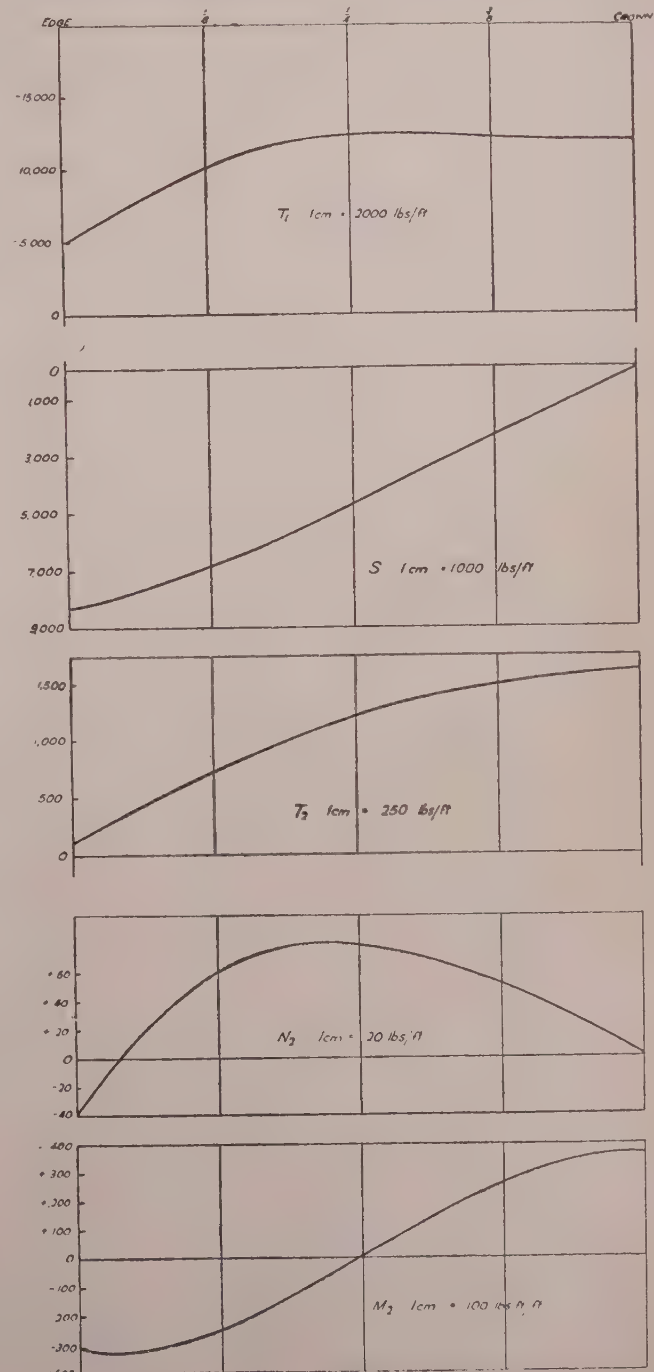


FIG 11.

GRAPH SHEWING DISTRIBUTION OF FORCES IN SHELL

decided. For this two objectives must be borne in mind: firstly, to be economical and, secondly, to try to maintain the shell in a state as near to the ideal as is consistent with economy.

The idealised design was based on many false, and some contradictory assumptions. Furthermore, the influences of such phenomena as the plasticity, shrinkage and creep of the concrete were ignored, as also was the effect of variations in the ambient temperature. Openings are frequently required in the shell for the purposes of lighting and ventilation, and these openings not infrequently effect a large proportion of the shell. Allowance for all these factors is largely a matter of experience, but certain effects of the redistribution of

DESIGN SHEET I

	1	Span $L = 78'\text{-}0''$	Chord Width $W = 28'\text{-}0''$	$\sin\phi = \frac{W}{2R} = \frac{28}{50} = \cdot 560$
	2	Radius $R = 25'\text{-}0''$	Thickness $d = 2\frac{1}{2}'' = \cdot 2085'$	$\phi = 34^\circ 03' = \cdot 594$ Rad.
SHELL PARAMETERS	3	$a = \left(\frac{R\pi}{L}\right)^2 = \left(\frac{\pi \times 25}{78}\right)^2 = 1\cdot 014$	$b = \frac{\sqrt{3}R}{d} = \frac{\sqrt{3} \times 25}{\cdot 2085} = 208$	$\cos\phi = \cdot 8285$ $\cos^2\phi = \cdot 686$

SHELL PARAMETERS	4	$r^2 = a \times b = 210\cdot 6$	$r = 14\cdot 51$	$\sqrt{r} = 3\cdot 81$	$r\sqrt{r} = 55\cdot 3$	$\sin\phi\cos\phi = \cdot 464$
	5	$\frac{1}{d^2} = \frac{1}{\cdot 2085^2} = 23\cdot 03$	$K_1 = 0\cdot 455\sqrt{r} = 1\cdot 732$		$K_1\phi_K = 1\cdot 732 \times 2 \times \cdot 594 = \underline{2\cdot 06}$	

MEMBRANE REACTIONS & DISPLACEMENTS A	6	LOAD: g = D/W + live + finish = 32 + 15 + 3 = 50 lbs/sq ft	$M = \frac{4}{\pi}\frac{L^2}{\pi^2} = \frac{4}{\pi}\frac{78^2}{\pi^2} = 785$
	7	$V^m = +g\,R\sin\phi\cos\phi = +50 \times 25 \times \cdot 464 = +580$ lbs/ft	
	8	$H^m = +g\,R\cos^2\phi = +50 \times 25 \times \cdot 686 = +858$ lbs/ft.	$\frac{ds^m}{dx} = +2g\sin\phi = -2 \times 50 \times \cdot 56 = +56$ lbs/sq.ft.
	9	$\delta_A^m = \frac{4g}{\pi d}\left(a\cos^2\phi + 4 + \frac{2}{a}\right) = \frac{4 \times 50}{\pi \times \cdot 2085}\left(1\cdot 014 \times \cdot 686 + 4 + \frac{2}{1\cdot 014}\right) = 305 \times (\cdot 696 + 4 + 1\cdot 97) = 305 \times 6\cdot 666 = \underline{2{,}040}$ lbs. ft. units.	
	10	$\delta_B^m = \frac{4g}{\pi d}\,a\cos\phi\sin\phi = \frac{4 \times 50 \times 1\cdot 014 \times \cdot 464}{\pi \times \cdot 2085} = \underline{144}$ lbs. ft. units.	
	11	$\delta_C^m = \frac{4}{\pi}\frac{L^2}{\pi^2}\frac{2g\cos\phi}{dR} = \frac{785 \times 2 \times 50 \times \cdot 8285}{\cdot 2085 \times 25} = \underline{12{,}470}$ lbs. ft. units.	

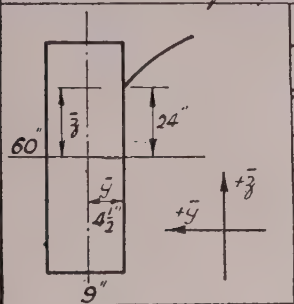
SHELL EDGE DISPLACEMENT CO-EFFICIENTS B.			Q		From Table I	× Q	× sin ϕ	× cos ϕ
	12	v_T	$\frac{1}{\sqrt{r}d^2}$	= 6·04	− 10·85	− 65·6	− 36·75 (a)	− 54·3 (f)
	13	$v_N = w_T$	$\frac{1}{d^2}$	23·03	+ 6·964	+ 160·5	+ 89·9 (b)	+ 133·0 (g)
	14	$v_S = \sigma_T$	$\frac{R}{rd^2}$	39·70	+ 7·042	+ 279·5	+ 156·7 (c)	+ 231·5 (h)
	15	w_N	$\frac{\sqrt{r}}{d^2}$	87·75	− 6·270	− 550·0	− 308·0 (d)	− 455·0 (j)
	16	$w_S = \sigma_N$	$\frac{R}{\sqrt{r}d^2}$	151·2	− 3·660	− 555·0	− 311·0 (e)	− 460·0 (k)
	17	σ_S	$\frac{R^2}{r\sqrt{r}d^2}$	260·5	− 6·002	− 1563		

		RESOLUTION FOR VERTICAL & HORIZONTAL FORCES.	
	18	$v_V = v_T\sin\phi - v_N\cos\phi = a - g = -36\cdot 75 - 133 = -169\cdot 75$ lbs/sq.ft.	
	19	$w_V = b - j = +89\cdot 9 + 455 = +544\cdot 9$ lbs/sq.ft.	$\sigma_V = -c + k = -156\cdot 7 - 460 = -616\cdot 7$ lbs/sq.ft.
	20	$v_H = -v_T\cos\phi - v_N\sin\phi = -f - b = +54\cdot 3 - 89\cdot 9 = -35\cdot 6$ lbs/sq.ft.	
	21	$w_H = -g - d = -133 + 308 = +175$ lbs/sq.ft.	$\sigma_H = +h + e = +231\cdot 5 - 311 = -79\cdot 5$ lbs/sq.ft.
		RESOLUTION FOR VERTICAL & HORIZONTAL DISPLACEMENTS	
	22	$\delta_{AV} = -\sin\phi \times v_V + \cos\phi \times w_V = \cdot 56 \times 169\cdot 75 + \cdot 8285 \times 544\cdot 9 = 95\cdot 1 + 451\cdot 5 = +\underline{546\cdot 6}$ lbs/sq.ft.	
	23	$\delta_{AH} = -\sin\phi \times v_H + \cos\phi \times w_H = \cdot 56 \times 35\cdot 6 + \cdot 8285 \times 175\cdot 0 = 19\cdot 9 + 145\cdot 1 = +\underline{165\cdot 0}$ lbs/sq.ft.	
	24	$\delta_{BV} = \cos\phi \times v_V + \sin\phi \times w_V = \cdot 8285 \times -169\cdot 75 + \cdot 56 \times 544\cdot 9 = -140\cdot 8 + 305\cdot 5 = +\underline{164\cdot 7}$ lbs/sq.ft.	
	25	$\delta_{BH} = \cos\phi \times v_H + \sin\phi \times w_H = \cdot 8285 \times -35\cdot 6 + \cdot 56 \times 175\cdot 0 = -29\cdot 5 + 98\cdot 0 = +\underline{68\cdot 5}$ lbs/sq ft.	
		N B As a check δ_{AH} should = δ_{BV}	
	26	Remaining Co efficients. $\delta_{CV} = \delta_{AS}\ \underline{-616\cdot 7}$. $\delta_{CH} = \delta_{BS} = \underline{-79\cdot 5}$. $\delta_{CS} = +\underline{1563}$ lbs/sq.ft.	

DESIGN SHEET II

C — BEAM CO-EFFICIENTS DISPLACEMENT

27 LOAD ON E.B. q = s/w + thickening = 565 + 150 = 715 lbs/ft.



28 Area A = 5 × ·75 = 3·75 sq. ft

29 $I_z = \frac{1}{12}\times\cdot75\times5^3 = 7\cdot81\ ft^4$ | $\bar{z}$ = = 2·0 ft.

30 $I_y = \frac{1}{12}\times5\times\cdot75^3 = \cdot1758\ ft^4$ | $\bar{y} = -4\frac{1}{2}'' = -\cdot375$ ft.

31 $\delta_{AV} = \frac{M}{I_z}$ = 785/7·81 = $\underline{100\cdot4}$ lbs/sq. ft.

32 $\delta_{CV} = \delta_{AV}\times\bar{z}$ = 100·4 × 2 = $\underline{200\cdot8}$ lbs/sq. ft. $= \delta_{AS}$

33 $\delta_{BH} = M/I_y$ = 785/·1758 = $\underline{4460}$ lbs/sq. ft. $\delta_{CH} = \delta_{BH}\times\bar{y}$ = −·375 × 4460 = $\underline{-1675}$ lbs/sq. ft. $= \delta_{BS}$

34 $\delta_{CS} = \frac{M}{A} + \delta_{CV}\times\bar{z} + \delta_{CH}\times\bar{y} = \frac{785}{3\cdot75} + 200\cdot3\times2 + 1675\times\cdot375 = 209\cdot3 + 401\cdot6 + 620 = \underline{1231}$ lbs/sq. ft.

D — EDGE MEMBRANE DISPLACEMENTS

35 $\delta_{A_m} = -(V^m+q)\,\delta_{AV} = (580+715)\,\delta_{AV}$ = −1295 × 100·4 = −130,000
$+\frac{ds}{dx}^m\,\delta_{AS}$ = +56 × 200·8 = +11,260 ∴ $\delta_A^m = \underline{-118{,}740}$ lbs/sq. ft

36 $\delta_{B_m} = H^m\,\delta_{BH}$ = 858 × 4460 = 3,830,000
$+\frac{ds}{dx}^m\,\delta_{BS}$ = − 56 × 1675 = − 93,800 ∴ $\delta_B^m = \underline{3{,}736{,}200}$ lbs/sq. ft

37 $\delta_{C_m} = -(V^m+q)\,\delta_{CV}$ = −1295 × 200·8 = − 260,000
$+H^m\,\delta_{CH}$ = −858 × 1675 = −1,435,000
$+\frac{ds}{dx}^m\,\delta_{CS}$ = +56 × 1231 = + 69,000 ∴ $\delta_C^m = \underline{-1{,}626{,}000}$ lbs/sq. ft.

E — EQUATIONS FOR V & H & $\frac{ds}{dx}$

	EQUATIONS		V	H	$\frac{ds}{dx}$	MEMBRANE m
38	SHELL		+ 546·6	+ 165·0	− 616·7	+ 2,040
39	EDGE BEAM	δ_A	+ 100·4	—	+ 200·8	− 118,740
40	GAP △	(i)	+ 647	+ 165·0	− 415·9	− 116,700
41	SHELL		+ 165·0	+ 68·5	− 79·5	+ 144
42	EDGE BEAM	δ_B	—	+ 4460	− 1675	+ 3,736,200
43	GAP △	(ii)	+ 165·0	+ 4528	− 1754	+ 3,736,000
44	SHELL		− 616·7	− 79·5	+ 1563	+ 12,470
45	EDGE BEAM	δ_C	+ 200·8	− 1675	+ 1231	− 1,626,000
46	GAP △	(iii)	− 415·9	− 1754	+ 2794	− 1,613,500

SOLUTION OF EQNS.

47	From i) + 647 V + 165·0 H − 415·9 S − 116,700 = 0	From iv)
48	From ii) + 647 V + 17,700 H − 6,885 S + 14,650,000 = 0	+ 2,567 H = − 2,155,000 + 946 × 158·5 / 150,000
49	iv) — + 17,535 H − 6,469 S + 14,767,000 = 0	= − 2,005,000
50	From iii) − 647 V − 2,732 H + 4,350 S − 2,512,000 = 0	∴ H = − 780 lbs/ft.
51	v) ∴ — − 2,567 H + 3,934 S − 2,629,000 = 0	From i)
52	From iv) + 2,567 H − 946 S + 2,155,000 = 0	$V = \frac{116{,}700 + 780\times165 + 415\cdot9\times158\cdot5}{647}$
53	— + 2,988 S − 474,000 = 0	$= \frac{116{,}700 + 128{,}800 + 65{,}500}{647} = \frac{311{,}000}{647}$
54	∴ $\frac{ds}{dx}$ = + 158·5 lbs/sq. ft.	V = + 481 lbs/ft.

forces can be foreseen. The three principal reinforcing systems—the longitudinal tension, the shear, and the bending reinforcement—will be considered in turn.

Main Tensile Reinforcement

The area of tensile reinforcement was obtained in the calculation by dividing the total tension in the edge beam by the working stress of the steel. Had the stress in the steel been assumed to be the modular ratio times the theoretical concrete stress approximately three times the area of steel would have been required. It is justifiable to use the smaller amount of reinforcement?

Fig. 12 shows some typical distributions of the longitudinal force T_1. Long shells with small unsupported edge beams and medium length to long internal shells, have a practically straight line distribu-

DESIGN SHEET III

55	$T_2 = V\sin\phi - H\cos\phi = 481 \times \cdot 56 + 780 \times \cdot 8285 = 270 + 646 = 916$ lbs/ft.					
56	$N_2 = -V\cos\phi - H\sin\phi = -481 \times \cdot 8285 + 780 \times \cdot 56 = -400 + 437 = +37$ lbs/ft.					
57	Factors Q	$M_2 = 1$	$T_2 = \frac{r}{R} = \cdot 58$	$N_2 = \frac{1\cdot 098\sqrt{r}}{R} = \cdot 1675$	$\frac{ds}{dx} = \frac{1\cdot 554 r\sqrt{r}}{R^2} = \cdot 1375$	$T_1 = \frac{2\sqrt{3}}{d} = 16\cdot 63$
58	Reduced Forces:	$T_2 \quad 916 \times \frac{\pi}{4} \times \frac{1}{\cdot 58} = 1241$ lbs/ft.		$N_2 = 37 \times \frac{\pi}{4} \times \frac{1}{\cdot 1675} = 173\cdot 5$ lbs/ft.		
59		$\frac{ds}{dx} = 158\cdot 5 \times \frac{\pi}{4} \times \frac{1}{\cdot 1375} = 905$ lbs/sq.ft.				

MEMBRANE DISTRIBUTION

		0	1/8	1/4	3/8	1/2
	Point	0	$\frac{1}{8}$	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{2}$
60	θ	34°-03′	25°-31′	17°-01′	8°-31′	0
61	$\sin\theta$	·5600	·4310	·2928	·1481	0
62	$\cos\theta$	·8285	·9023	·9562	·9890	1·0
63	$T_2 = -qR\cos\theta = -25 \times 50\cos\theta = -1250\cos\theta$	−1036	−1128	−1195	−1236	−1250
64	$\frac{ds}{dx} = 2q\sin\theta = 2 \times 50\sin\theta = 100\sin\theta$	+ 56	+ 43·1	+ 29·28	+ 14·81	0
65	$T_1 = -\frac{2L^2 q\cos\theta}{\pi^2 R} = \frac{2 \times 78^2 \times 50\cos\theta}{\pi^2 \times 25} = -2465\cos\theta$	−2040	−2224	−2356	−2436	−2465

DISTRIBUTION OF FORCES IN SHELL

			Coeff.	0	Coeff.	1/8	Coeff.	1/4	Coeff.	3/8	Coeff.	1/2
F												
66	T_2	+1241	+2·036	+2525	+·5822	+ 722	−·2947	− 366	−·7237	− 903	−·8564	−1063
67	N_2	+173·5	−1·162	− 202	−·2220	− 39	+·2026	+ 35	+·3253	+ 56	+·3411	+ 59
68	$\frac{ds}{dx}$	+ 905	−2·692	−2438	−1·1549	−1046	−·1386	−126	+·4389	+397	+·6241	+564
69	Σ			−115		−363		−457		−450		−440
70	× Q	× 16·63		−1915		−6040		−7610		−7490		−7320
71	+ m			−2040		−2224		−2356		−2436		−2465
72	Total			−3955		−8264		−9966		−9926		−9785
73	× $\frac{\pi^2}{8}$	∴	$\boxed{T_1 =}$	−4860		−10,200		−12,300		−12,230		−12,070
74	T_2	+1241			+·914	+1137	+·987	+1226	+·5914	+ 733		
75	N_2	+173·5			−·467	− 81	−·448	− 78	−·2424	− 42		
76	$\frac{ds}{dx}$	+905			−·099	− 90	−·544	−492	−·4096	−371		
77	Σ					+966		+656		+ 320		
78	× Q	× ·1375		+158·5		+132·8		+ 90·2		+44·0		
79	+ m			+ 56		+ 43·1		+ 29·3		+14·8		
80	Total			+214·5		+175·9		+119·5		+58·8		
81	× $\frac{3}{2}$	× 39	$\boxed{S =}$	+8360		+6860		+4660		+2290		
82	T_2	+1241			+·794	+ 985	−·088	− 109	−·805	− 998	−1·072	−1330
83	N_2	+173·5			+·255	+ 44	+·679	+ 118	+·989	+ 172	+1·097	+ 190
84	$\frac{ds}{dx}$	+905			−·4358	−394	−·097	− 88	+·3525	+ 317	+·542	+ 492
85	Σ					+635		− 79		− 509		− 648
86	× Q	× ·58		+ 916		+368		− 46		− 295		− 376
87	+ m			−1036		−1128		−1195		−1236		−1250
88	Total		$\boxed{T_2 =}$	−120		− 760		−1241		−1531		−1626

tion (similar to that of a long beam) as in (a). The maximum values of $K_1 \cdot \Phi_k$ for this type of distribution are about 2.5 for external half shells (the value depending on the depth of the edge beam) and 3.0 for internal shells.

Shells slightly shorter than as above and long shells with deep edge beams have a distribution which deviates from a straight line as in (b). Still shorter shells have distributions as in (c) and (d), the values of $K_1 \ \Phi_k$ corresponding to these latter distributions for internal feather-edged shells are about 3.3 and 3.7 respectively. Fig. 12 (e) shows the distribution of T_1 for a shell in which the concrete is assumed to have no strength in tension, and such that the elastic distribution is as in (a).

For simplicity feather-edged internal shells are considered; similar reasoning can be applied to the external half-barrels bearing in mind the type of stress distribution appropriate to them. Considering first long shells, the concrete tensile stress will be greater than the concrete can carry so that a cracked section will be

DESIGN SHEET IV

DISTRIBUTION OF FORCES (CONTD.)

			COEF.	0	COEF.	$\frac{1}{8}$	COEF.	$\frac{1}{4}$	COEF.	$\frac{3}{8}$	COEF.	$\frac{1}{2}$
89	T_2	+1241			-.5665	-703	-.751	-932	-.5056	-627		
90	N_2	+173.5			+1.225	+213	+.9837	+171	+.546	+95		
91	$\frac{ds}{dx}$	+905			+.1636	+148	+.3173	+287	+.246	+224		
92	Σ					-342		-474		-308		
93	×Q	×.1675	N_2 =	+37		-57.4		-79.4		-51.6		0
94	T_2	+1241	+.600	+744	+.4115	+510	-.0119	-14.8	-.422	-523	-.586	-728
95	N_2	+173.5	-1.341	-233	-.5578	-97	+.1386	+24.1	+.624	+108	+.798	+138
96	$\frac{ds}{dx}$	+905	-.2146	-195	-.1747	-158	-.0159	-14.4	+.172	+156	+.254	+230
97	Σ		M_2 =	+316		+255		-5.1		-259		-360

CHECKS

CHECKS ON EQUILIBRIUM.

	Pt.	λ	T_1	λT_1	(cos φ - .8285)	y = R(cos φ - .8285)	$\lambda T_1 y$	S	λS	sin φ	λS sin φ
98	0	1	-4,860	-4,860	0	0	- 0	8360	+8,360	.56	4,680
99	$\frac{1}{8}$	4	-10,200	-40,800	.0738	1.845	- 75,300	6860	+27,440	.431	11,810
100	$\frac{1}{4}$	2	-12,300	-24,600	.1277	3.19	- 78,500	4660	+9,320	.2928	2,730
101	$\frac{3}{8}$	4	-12,230	-48,920	.1605	4.01	-196,000	2290	+9,160	.1481	1,360
102	$\frac{1}{2}$	1	-12,070	-12,070	.1715	4.29	- 51,800	0	0	0	0
103			Σ =	131,250			401,600		54,280		20,580
104	$\times \frac{.594 \times 25}{12} = 1.238$ =			162,500 lbs			497,000 lbs ft.		67,200 lbs		25,450 lbs

105 Tension in Edge Beam $= S_0 \times \frac{L}{4} = 8360 \times \frac{78}{4}$ = 163,000 lbs

106 Bending Moment @ $\frac{wL^2}{8} = \frac{78^2}{8}(50 \times 25 \times .594 + 481 - 580) = \frac{78^2}{8} \times (743 - 99) = \frac{78^2}{8} \times 644$ = 490,000 lbs. ft.

107 Total Load $= \frac{wL}{2} = 644 \times 39$ = 25,100 lbs

REINFORCEMENT

REINFORCEMENT

108 1) Shear: $A_S = \frac{67,200}{18,000\sqrt{2}} = 2.63$ sq. in
Say 25 - $\frac{3}{8}''\phi$

Min. Spacing: $\frac{8360}{18000\sqrt{2}} = .328$ sq. in./ft.
$\frac{3}{8}''\phi$ @ 4" crs

Bending: M_{max} = 360 lbs. ft/ft. = 4320 lbs. in/ft.

$A_T = \frac{4320}{27,000 \times .88 \times 1.7} = .1070$ sq in./ft.

Mesh with 2ga. @ 6" crs.

EDGE BEAM.

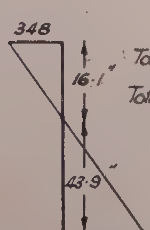
Load = 715 + 99 = 814 lbs/ft. B.M. $= 814 \times \frac{78^2}{8} = 618,000$ lbs ft.

B.M. DUE TO SHEAR = $-163,000 \times 2 = -326,000$ lbs ft.

NET B.M = 292,000 lbs ft.

STRESS $= \frac{T}{A} \pm \frac{M}{Z} = \frac{163,000}{9 \times 60} \pm \frac{292,000 \times 6 \times 12}{9 \times 60^2} = 302 \pm 650$ lbs/sq. in.

i.e. + 952 & -348 lbs/sq. in



Total Comp. $= \frac{1}{2} \times 9 \times 16.1 \times 348 = -$ 25,200 lbs

Total Tension $= \frac{1}{2} \times 9 \times 43.9 \times 952 = +$ 188,000 lbs

Nett Tension 162,800 lbs. (Check.)

$A_T = \frac{188,000}{27,000}$ = 6.98 sq. ins. Sq. Twisted.

formed. The lever arm will thus be increased and the tensile force required less than that assumed in the design.

Short shells with distributions as in (c) and (d) have a high rise/span ratio (about 1/4 to 1/3) and the concrete stresses are low. Due to the design forces the concrete should not crack so that the steel supplied will have a very low stress. Due to shrinkage and temperature variations the shell forces may be radically altered and the concrete section crack. In this case, since the reinforcement is designed on the basis of an uncracked section which has a smaller lever arm than a cracked section the quantity of reinforcement is sufficient. In roofs of this type it is important to provide longitudinal reinforcement throughout the shell to guard against possible cracking due to shrinkage, temperature or the effect of restraints provided by any other part of the structure.

If a reinforced concrete beam is tested and the strains measured, using an 8 in. or 10 in. gauge length, on the face of the concrete at the same level as the reinforcement it is found that the actual strain is considerably less than the theoretical strain based on a uniform tension of the reinforcement within the gauge length. This is due to the fact that whilst at a cross-section where the concrete is cracked there is the theoretical stress the bond of the concrete is sufficient to reduce appreciably the stress in the steel between the cracks, causing a reduction in the strain. The discrepancy between the assumed steel stress and the modular ratio times the concrete stress does not, therefore, cause as great a redistribution of the forces as, at first sight, might be expected.

There is an additional factor to be considered with narrow shells and shells with low rise/span ratio. This is the plastic deformation and creep of the concrete under the high stresses obtained. The longitudinal stress distribution is generally as in Fig. 12 (a) and after plasticity and creep the redistribution is as in (b), the centre of compression is lowered and thus more reinforcement is required than is shown by the calculations. This increase is offset by the increase in the lever arm due to a cracked section being formed.

A further feature of roofs with a low rise/span and relatively large radius is that the forces in the shell can cause considerable deformations. If the radius of curvature of the shell is decreased the strength of the shell is increased, but if the radius of curvature is increased, i.e., the shell flattens, the problem of elastic instability arises. Since the deformations due to creep and plasticity may be of the same order as or even greater than the elastic displacements, it is advisable either to avoid low rise/span ratios or else to reduce the longitudinal stresses by means of prestress. With a rise/span ratio of 0.1 and a chord width/span ratio of .4 to .5 the longitudinal stresses are reasonable for spans of up to 110 ft. or 120 ft. With larger spans and with shells which have lower values for the above-mentioned ratios prestressing is advisable.

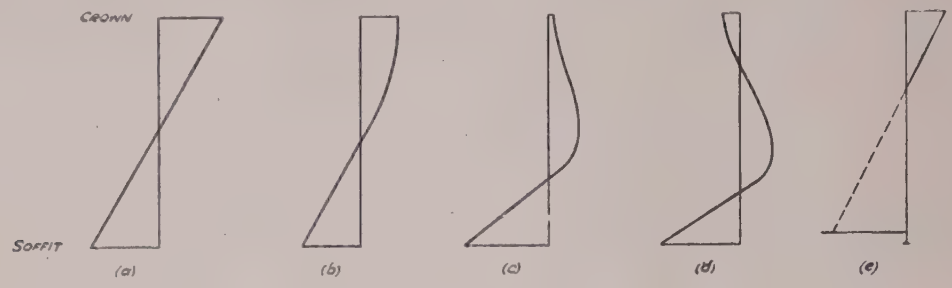


FIG 12 TYPICAL DISTRIBUTIONS OF T_1 FORCES.

Shear Reinforcement

The distribution of the shear forces in the shell depends on the distribution of the longitudinal forces T_1. The shear distributions corresponding to the T_1 diagrams above are shown in Fig. 13. Considering first cases (a) and (e), i.e., the " uncracked " beam and the " cracked " beam distributions, since the sum of the vertical components of the shear forces is constant (equal to the applied load) and since in the distribution (e) a greater proportion of the shear occurs in the lower part of the structure where the shell is steeper, both the total shear and the maximum shear stress are reduced when a cracked section is assumed.

At the other end of the scale, however, a distribution as in (d) already has the majority of the shear force in the steeper part of the shell and a redistribution of the forces will increase the total shear with little reduction in the maximum value of the shear stress. Thus in shells with distributions of shear as in (c) and (d) it is

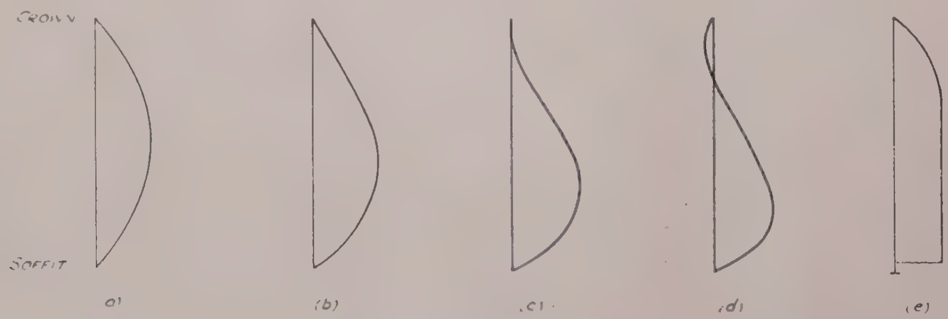


FIG 13 TYPICAL DISTRIBUTIONS OF SHEAR FORCES

preferable to provide shear reinforcement up to the crown of the shell although this is not theoretically required.

The manner in which the shell is reinforced for shear can be in various forms. In the earlier days of shell construction the principal stress trajectories were found and the reinforcement provided following these lines. These were, needless to say, of a complicated shape and more rational methods followed, consisting of either diagonal bars across the corners (equivalent to bent up bars in beams) or circumferential bars (equivalent to stirrups). The author prefers the former system (diagonal bars) as these provide a definite tie between the shell and the supporting structure, i.e., the stiffening beams. The joint between the shell and the stiffening beams is generally an "awkward" spot as there is a rather heavy concentration of reinforcement and frequently a construction joint as at the junction.

Bending Reinforcement

As has been mentioned earlier, the shell has been designed on the assumption of no rotation at the edge beam, and this assumption affects the bending moments in the shell. Further, the various factors such as thermal variations, creep, etc., cause more variations in M_2 than in the other forces. For these reasons the values of M_2 found in the calculations should only be regarded as a guide to the magnitude of the moments to be expected. To minimise the effects of temperature variations it is advisable to provide thermal insulation on the exterior surface of the shell.

The shell should be thickened at the edges, the amount varying with the span and radius of the shell, to the order of 6 in. to 9 in. for normal sizes. Large positive bending moments should be avoided as far as is possible as these tend to flatten the shell.

Deflections

The deflections of the structure should also be considered. These were not shown in the calculation but can easily be found from the influence coefficients and the redundant forces, i.e., if δ_{AX} is the vertical deflection due to force X then :

$$\delta_A = \frac{L^2}{\pi^2 E} \Sigma X \delta_{AX}$$

and the resultant deflection added to the membrane deflection.

In the example the vertical deflection at the edge is thus :

$$\delta_A = \frac{78^2 \times 12 \left\{ (481 \times 546.6 - 780 \times 165 - 158.5 \times 616.7) + 2040 \right\}}{\pi^2 \times 3{,}000{,}000 \times 144} = 0.66 \text{ ins.}$$

(E is assumed to be 3,000,000 lb. per sq. inch.)

The theoretical value of the deflection is reasonably reliable for shells where the compressive stress is not high, but for larger shells where the stresses are high the deflection due to plasticity and creep may be two or three times the value found by this method, as with normal reinforced concrete work.

Owing to the uncertainty of the actual deflection to be expected, particularly with larger shells, care should be exercised to provide sufficient freedom for the shell to move with respect to the rest of the structure, e.g., partition walls, etc. For external edges of buildings where the concrete line is required to conform to the general lines of the building, it is advisable, as well as more economical, to provide columns to support the edge beams.

Conclusion

The value of calculations of the type outlined for barrel vault roofs can be summarised thus :

1. The distribution of the T_1 forces in the shell gives an overall picture of the shell as a whole and enables the quantity of reinforcement that is required to provide a sound economical structure to be determined.

2. The shear force distribution not only gives a basis of design for the quantity of reinforcement which is essential, but indicates the need for further reinforcement on occasions when this is advisable.

3. The bending moment M_2 distribution provides a guide to the magnitude of the moments to be expected, and gives a warning when unsuitable structures are considered.

Acknowledgements

The author wishes to express his thanks to Messrs. "Twisteel" Reinforcement, Ltd., and their associated company, Messrs. Barrel Vault Roofs (Designs) Ltd., for permission to draw upon their experience gained in the design of a large number of shell structures, and for their generous assistance in the preparation of the tables presented here.

APPENDIX I

Summary of Notation

A	cross-sectional area of edge beam
A	as suffix denotes vertical direction
B	as suffix denotes horizontal direction
C	as suffix denotes longitudinal stress
E	Youngs Modulus
H	Horizontal Force
	as suffix denotes : due to unit horizontal force
I	Moment of Inertia
I_y	Moment of Inertia of edge beam about vertical axis through C of G
I_z	Moment of Inertia of edge beam about horizontal axis through C of G
K_1	Shell parameter $= .455 \sqrt{r}$
L	Length of shell
M	Moment due to unit load $= \frac{4}{\pi} \cdot \frac{L^2}{\pi^2}$. In Appendix II—Factor of M_2
M_2	Bending moment in shell (Fig. 5)
N_2	radial shear force in shell (Fig. 5)
Q	multiplier for quantity as indicated by suffix
R	radius of shell
S	Shear force in shell (Fig. 5)
T	Tensile force applied to edge beam
T_1	Longitudinal tensile force in shell (Fig. 5)
T_2	Circumferential tensile force in shell (Fig. 5)
V	Vertical force
	as suffix denotes : due to unit vertical force
a	shell parameter $= \left(\frac{\pi R}{L}\right)^2$
b	shell parameter $= \frac{\sqrt{3}R}{d}$
d	thickness of shell
g	load on shell/unit area
h	rise of shell from springing line
m	denotes membrane condition
	as suffix : applies to edge beam
m	as superfix applies to shell
q	load on edge beam
r	shell parameter $= \sqrt{a\ b}$
u	longitudinal displacement
v	tangential displacement

w	radial displacement
x	longitudinal direction
y	horizontal direction for edge beam
$\bar{y}$	horizontal distance of springing from C of G of edge beam
z	vertical direction for edge beam
$\bar{z}$	vertical distance of springing from C of G of edge beam
β	shell parameter $= \sqrt{2}\ a\ b$
γ	shell parameter $= \sqrt{\frac{a}{b}}$
δ	displacement
δ_A	displacement in direction A, etc.
$\delta^1{}_A$	displacement of edge beam in direction A, etc.
θ	angular distance from crown in membrane condition
Φ	half angle subtended by shell
Φ_K	central angle of shell
σ	longitudinal stress
$\triangle$	gap between shell and edge beam with suffix : change of gap
ψ	Change of slope of shell

N.B. In Appendix II

A, B, C, D Arbitrary constants of integration

a, b, c, d, constants depending on shell dimensions

α, β, γ, δ, constants depending on force considered

APPENDIX II

A. MEMBRANE FORCES AND DISPLACEMENTS

Consider the cylindrical shell with dimensions shown in Fig. 4.

Let the applied load normal to surface $= Z$/unit area

,, ,, ,, tangential to ,, $= Y$/unit area

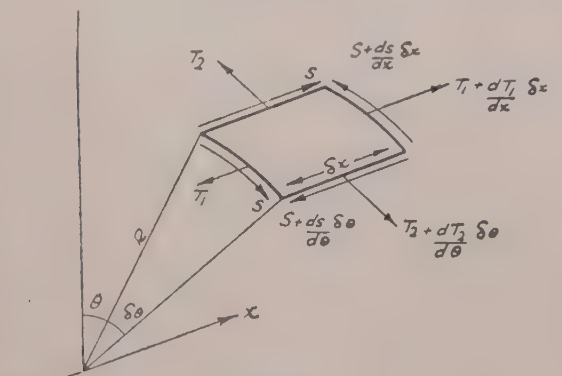


FIG. 14

I. FORCES

For an element of the shell at an angle θ from the crown (Fig. 14) we have for equilbrium :

(i) Resolving normally to surface

$$T_2\ \delta x\ \frac{\delta\theta}{2} + \left(T_2 + \frac{dT_2}{d\theta}\delta\theta\right)\delta x\frac{\delta\theta}{2} + ZR\ \delta\theta\ \delta x = 0$$

whence $T_2 = RZ$ . . . . (11)

(ii) Resolving tangentially :

$$T_2\,\delta x - \left(T_2 + \frac{dT_2}{d\theta}\delta\theta\right)\delta x - SR\delta\theta + \left(S + \frac{dS}{dx}\delta x\right)R\delta\theta - YR\delta\theta\delta x = 0$$

whence $\frac{ds}{dx} = \frac{1}{R}\frac{dT2}{d\theta} + Y = \frac{dZ}{d\theta} + Y$ (12)

(iii) Resolving longitudinally :

$$T_1R\delta\theta - \left(T_1 + \frac{dT_1}{dx}\delta x\right)R\delta\theta - S\delta x + \left(S + \frac{dS}{d\theta}\delta\theta\right)\delta x = 0$$

whence $\frac{dT1}{dx} = \frac{1}{R}\frac{dS}{d}$ $\therefore$ $\frac{d^2T_1}{dx^2} = \frac{1}{R}\frac{d^2S}{dxd\theta}$

$$= \frac{1}{R}\frac{-d^2Z}{d\theta^2} + \frac{dY}{d\theta} \quad . . . . (13)$$

Taking a vertical load of g/unit area of surface of the shell and representing this by a Fourier series in the direction of x :

$$g = \frac{4}{\pi}g\left\{\cos\frac{\pi x}{L} - \frac{1}{3}\cos\frac{3\pi x}{L} + \ldots\right\}$$

Considering only the first term $g = \frac{4}{\pi}g\cos\frac{\pi x}{L}$

$$Z = \frac{4}{\pi}g\cos\theta\cos\frac{\pi x}{L},\quad Y = \frac{4}{\pi}g\sin\theta\cos\frac{\pi x}{L}$$

whence from equations (11)—(13).

$$T_2 = -\frac{4}{\pi}g\ R\cos\theta\cos\frac{\pi x}{L} \quad . . . . . . (14)$$

$$\frac{dS}{dx} = \frac{4}{\pi}2g\sin\theta\cos\frac{\pi x}{L} \quad . . . . (15)$$

Integrating (13) :

$$T_1 = -\frac{4}{\pi}\frac{L^2}{\pi^2}\frac{2g}{R}\cos\theta\cos\frac{\pi x}{L} \quad . . . . (16)$$

in particular, when $x = 0$, $\cos\frac{\pi x}{L} = 1$, and equations (1)—(3) are obtained.

II. DISPLACEMENTS

Considering first the longitudinal strain ϵ_1 an element of length dx has a displacement of u at $x = x$ and $u + \frac{du}{dx}\delta x$ at $x = x + \delta x$

The change in length is therefore du and the strain $= \frac{du}{dx}$

The stress is T_1/d, thus $\epsilon_1 = \frac{du}{dx} = \frac{T_1}{Ed}$. (17)

The tangential strain consists of two parts.

If (Fig. 15) the original position of an element of the shell, length ds, is AB, and after straining is $A^1\,B^1$

(i) Due to radial movement of the element its length is reduced by an amount of $\frac{w\,ds}{R}$

(ii) Due to the circumferential movement of the element its length is increased by $\frac{dv}{d\theta}\,d\theta$

Thus the change in length, putting $ds = Rd$ is $\frac{dv}{d\theta}\,\delta\theta - w\,\delta\theta.$

and the strain $\varepsilon_2 = \frac{1}{R}\left(\frac{dv}{d\theta} - w\right) = \frac{T_2}{Ed}$ (18)

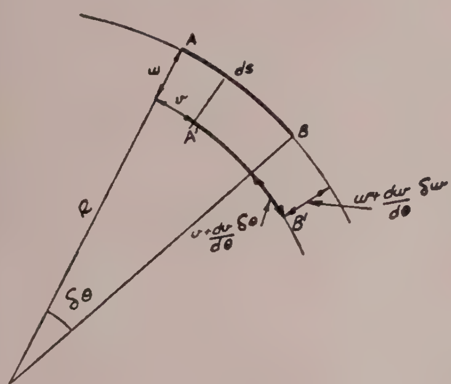


Fig. 15

The shear strain (Fig. 16) is $\frac{1}{R}\frac{du}{d\theta} + \frac{dv}{dx}$

and since we have taken the positive direction of S as increasing the angle at C

$$\omega = \frac{1}{R}\frac{du}{d\theta} + \frac{dv}{dx} = -\frac{2S}{Ed} \quad \ldots \ldots \ldots (19)$$

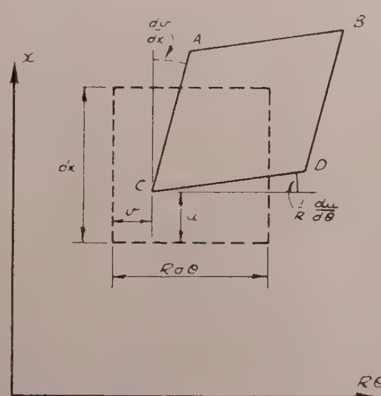


FIG 16

Differentially with respect to x

$$\frac{1}{R}\frac{d^2u}{dxd\theta} + \frac{d^2v}{dx^2} = -\frac{2}{Ed}\frac{dS}{dx}$$

from (15), (16) and (17)

$$\frac{d^2v}{dx^2} = -\frac{4}{\pi}\frac{g}{Ed}\left(\frac{2L^2}{\pi^2R^2} + 4\right) \sin\theta \cos\frac{\pi x}{L} \quad (20)$$

hence

$$v = \frac{4L^2g}{\pi\pi^2\,Ed}\left(\frac{2L^2}{\pi^2R^2} + 4\right) \sin\theta \cos\frac{\pi x}{L}$$

from (17), (14) and (20)

$$w = \frac{4L^2g}{\pi\pi^2\,Ed}\left(\frac{\pi^2R^2}{L^2} + 4 + \frac{2L^2}{\pi^2R^2}\right) \cos\theta \cos\frac{\pi x}{L} \quad (21)$$

multiplying by $\frac{\pi^2E}{L^2}$ and resolving vertically

$$\delta_A = v \sin\Phi - w\cos\Phi$$

$$\delta_A = \frac{4g}{\pi d}\left(a\cos^2\Phi + 4 + \frac{2}{a}\right)\cos\frac{\pi x}{L}$$

$$\left(a = \frac{\pi^2R^2}{L^2}\right) \quad \ldots \ldots \ldots (22)$$

$$\delta_B = w\sin\Phi - v\cos\Phi$$

$$\delta_B = \frac{4g}{\pi d}\,a\cos\Phi\sin\Phi\cos\frac{\pi x}{L} \quad \ldots \ldots (23)$$

At $x = 0$ the membrane displacements are given as in equations I and II.

B. GENERAL THEORY OF BENDING OF SHELLS

Assumptions :

(i) The shell thickness is small compared with its radius and length.

(ii) The twisting moments and the radial shear and bending moment on the transverse section are negligible.

(iii) The tangential strain ε_2 is small compared with the "radial strain" w/R.

(iv) The shear strain is small compared with the derivatives of u and v (see later).

(v) The shell is free of surface loads.

On the basis of these assumptions we have five forces in the shell —T_1, T_2, M_2, N_2 and S (Fig. 5) and these, together with the displacements u, v, w and ψ (change of slope), can all be expressed in terms of M_2 and its derivatives. From these can be built up for M_2 a partial differential equation of the eighth order with respect to end of the fourth order with respect to x.

A solution of the form $M_2 = \sum_{n=1,3,5\ldots} M_n \cos\frac{n\pi x}{L}$

where M_n is a function of θ only is admissible, and for simplicity this is assumed initially. Further, only the first term ($n = 1$) is considered. Terms of higher order ($n = 3, 5, \ldots$) can be considered in a similar manner to that outlined below.

Consider the equilibrium of an element of the shell as shown in Fig. 17.

Taking moments about longitudinal axis AB

$$\frac{dM_2}{d\theta}\,\delta\theta\,\delta x - N_2\,\delta x\,R\,\delta\theta = 0$$

Hence
$$N_2 = \frac{1}{R}\frac{dM_2}{d\theta} \quad \ldots \ldots (24)$$

Resolving in radial direction

$$T_2\,\delta\theta\,\delta x + \frac{dN_2}{d\theta}\,\delta\theta\,\delta x = 0$$

$$T_2 = -\frac{dN_2}{d\theta} = -\frac{1}{R}\frac{d^2M_2}{d\theta^2} \quad . \quad . \quad . \quad (25)$$

resolving tangentially

$$\frac{dT_2}{d\theta}\,\delta\theta\delta x + \frac{dS}{dx}\,R\delta\theta\delta x - N_2\delta\theta\delta x = 0$$

$$\frac{dS}{dx} = \frac{1}{R}\left(N_2 - \frac{dT_2}{d\theta}\right)$$

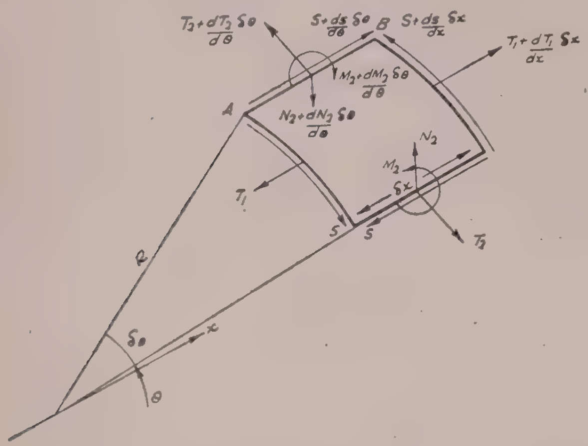


Fig. 17

It can be shown that the term $\frac{N_2}{R}$ is a second order term for thin shells and should, for consistency, be neglected.

Whence $$\frac{dS}{dx} = -\frac{1}{R}\frac{dT_2}{d\theta} = \frac{1}{R^2}\frac{d^3M_2}{d\theta^3} \quad . \quad . \quad (26)$$

Resolving longitudinally

$$\frac{dT_1}{dx}\,\delta x\, R\delta\theta + \frac{dS}{d\theta}\,\delta\theta\delta x = 0$$

hence

$$\frac{d^2T_1}{dx^2} = -\frac{1}{R}\frac{d^2S}{dxd\theta} = -\frac{1}{R^3}\frac{d^4M_2}{d\theta^4}$$

Putting $M_2 = M\cos\frac{\pi x}{L}$ and integrating,

$$T_1 = \frac{L^2}{\pi^2R^3}\frac{d^4M}{d\theta^4}\cos\frac{\pi x}{L} \quad . \quad . \quad . \quad . \quad . \quad (27)$$

from (17) and integrating

$$u = \frac{L^3}{\pi^3R^3Ed}\frac{d^4M}{d\theta^4}\sin\frac{\pi x}{L}$$

the shear strain (19) $\omega = \frac{1}{R}\frac{du}{d\theta} + \frac{dv}{dx}$

and on the basis of assumption (iv) it is assumed that each of the terms $\frac{1}{R}\frac{du}{d\theta}$ and $\frac{dv}{dx}$ is numerically large compared with ω.

This can be done justifiably for the general run of barrel vault roofs where the ratio $\frac{\text{Overall rise}}{\text{span}}$ is of the order of $\frac{1}{10}$

In short shells, however, this ratio is generally greater than unity and considerable inaccuracies would be expected to arise from this assumption. For the intermediate range of shells with a $\frac{\text{rise}}{\text{span}}$ ratio of, say, up to 1/3, it has been found in practice that reasonable results are obtained.

Thus $$\frac{dv}{dx} = -\frac{1}{R}\frac{du}{d\theta} = -\frac{L^3}{\pi^3R^4Ed}\frac{d^5M}{d\theta^5}\sin\frac{\pi x}{L}$$

whence $$v = \frac{L^4}{\pi^4R^4\,Ed}\frac{d^5M}{d\theta^5}\cos\frac{\pi x}{L} \quad . \quad . \quad . \quad . \quad (28)$$

from (18) on the basis of assumption (iii) we have

$$w = \frac{dv}{d\theta} = \frac{L^4}{\pi^4R^4Ed}\frac{d^6M}{d\theta^6}\cos\frac{\pi x}{L} \quad . \quad . \quad . \quad . \quad (29)$$

The change of slope of the shell $\psi = \frac{1}{R}\frac{dw}{d\theta}$

$$\psi = \frac{L^4}{\pi^4R^5Ed}\frac{d^7M}{d\theta^7}\cos\frac{\pi x}{L} \quad . \quad . \quad . \quad . \quad (30)$$

and the bending moment is given by

$$\frac{M_2}{EI} = -\frac{1}{R}\frac{d\psi}{d\theta} \text{ where } I = \frac{d^3}{12}$$

Hence $$\frac{M}{EI}\cos\frac{\pi x}{L} = -\frac{L^4}{\pi^4R^6Ed}\frac{d^8M}{d\theta^8}\cos\frac{\pi x}{L}$$

$$\frac{d^8M}{d\theta^8} + \frac{12\pi^4\,R^6}{L^4\,d^2}M = 0 \quad . \quad . \quad . \quad . \quad (31)$$

or $$\frac{d^8M}{d\theta^8} + 4a^2b^2 = 0 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (32)$$

where $a = \left(\frac{\pi R}{L}\right)^2$ and $b = \frac{\sqrt{3}\,R}{d}$

This is the fundamental differential equation for bending of shells due to Schorer. The theory above is based on that of Schorer but the assumptions implicit in his approximation are stated and are introduced into the equations as they occur. The remainder of the appendix is essentially an outline of Schorer's method.

To find a solution to the equation let $M = Ae^{m\theta}$

Hence $m^8 + 4\,a^2b^2 = 0 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (33)$

Putting $r^2 = ab$ $\quad m = 4\sqrt{2}\,\sqrt{r}\,\sqrt[8]{-1}$

There are eight values of $\sqrt[8]{-1}$ and these can be found by means of the Argand diagram, and are

$$\pm\cos 22\tfrac{1}{2} \pm i \sin 22\tfrac{1}{2} = \pm 0.9239 \pm i\, 0.3827$$
$$\pm\sin 22\tfrac{1}{2} \pm i \cos 22\tfrac{1}{2} = \pm 0.3827 \pm i\, 0.9239$$

since $e^{a+ib} = e^a (\cos b + i \sin b)$.

We have a solution in the form

$$M = e^{\pm J_1\theta}(A \sin K_1\theta + B \cos K_1\theta) + e^{\pm K_1\theta}(C \sin J_1\theta + D \cos J_1\theta) \quad . \quad . \quad (34)$$

where $4\sqrt{2}\,\sqrt{r}\,.9239 = 1.098\sqrt{r} = n_1\sqrt{r} = J_1$

$4\sqrt{2}\,\sqrt{r}\,.3827 = .455\sqrt{r} = n_2\sqrt{r} = K_1$

and since if we have a very large central angle to the shell the effect of M on one side dies away, the terms in $e^{+J_1\theta}$ and $e^{+K_1\theta}$ are equal to zero whence

$$M = e^{-J_1\theta}(A \sin K_1\theta + B \cos K_1\theta) + e^{-K_1\theta}(C \sin J_1\theta + D \cos J_1\theta) \quad . \quad . \quad . \quad (35)$$

Where A, B, C and D are arbitrary constants to be determined by boundary conditions.

It will be seen that the successive derivations of M will produce similar terms with varying coefficients, so putting

$$\begin{aligned} M = {} & Ae^{-J_1\theta}(\alpha \sin K_1\theta + \beta \cos K_1\theta) \\ & + Be^{-J_1\theta}(\alpha \cos K_1\theta - \beta \sin K_1\theta) \\ & + Ce^{-K_1\theta}(\alpha \sin J_1\theta + \delta \cos J_1\theta) \\ & + De^{-K_1\theta}(\gamma \cos J_1\theta - \delta \sin J_1\theta) \quad . \quad . \quad . \quad . \quad (36) \end{aligned}$$

where for M_2; $\alpha = \gamma = 1$; $\beta = \delta = 0$.

Now $N_2 = \dfrac{1}{R}\dfrac{dM_2}{d\theta}$, and, putting $K_1 = k J_1$, $(k = .4142)$

$$\begin{aligned} N_2 = \frac{J_1}{R}\Big\{ & Ae^{-J_1\theta}(-\sin K_1\theta + k \cos K_1\theta) \\ & + Be^{-J_1\theta}(-\cos K_1\theta - k \sin K_1\theta) \\ & + Ce^{-K_1\theta}(-k \sin J_1\theta + \cos J_1\theta) \\ & + De^{-K_1\theta}(-k \cos J_1\theta - \sin J_1\theta)\Big\} \quad . \quad . \quad . \quad (37) \end{aligned}$$

i.e., for N_2, $\alpha = -1$, $\beta = +k$, $\gamma = -k$, $\delta = +1$, and the general multiplier $\dfrac{J_1}{R} = 1.098\dfrac{\sqrt{r}}{R}$ is termed Q_{N_2}.

using equations (25) to (30) we get the values of α, β, γ, δ and Q, shown in Table III.

TABLE III

Quantity	Q	α	β	γ	δ	Quantity	Q	α	β	γ	δ
M_2	1	1	0	1	0	T_1	$\frac{2\sqrt{r}}{a}$	[illegible]	-1	0	+1
N_2	$\frac{1.098\sqrt{r}}{R}$	-1	+k	-k	+1	v	$\frac{2.96\, r^{2}\sqrt{r}}{Ea\,r^2}$	+k	+1	-1	-k
T_2	$\frac{r}{R}$	-1	+1	+1	+1	w	$\frac{2r^{3}}{Eaa^{2}}$	-1	-1	+1	-1
$\frac{dS}{dx}$	$\frac{.554\, r\sqrt{r}}{R^2}$	-k	+1	+1	-k	ψ	$\frac{3.108\, r^{3}\sqrt{r}}{Eaa^{2}R}$	+1	+k	+k	+1

Thus the values of the eight quantities M_2 N_2 T_2 T_1 $\dfrac{dS}{dx}$ v w and ψ are obtained in the terms of the four arbitrary constants A, B, C and D and the shell dimensions. Thus for any given values of four of these quantities, A, B, C and D can be found and hence the remaining four quantities.

Considering now a shell with central angle Φ_k with a moment of $M_2 = M_o \cos \dfrac{\pi x}{L}$ applied at each edge.

At the right-hand edge $\theta = 0$ an applied moment of M gives

$$M = B + D$$

At the left-hand edge $\theta = \Phi_k$ and a moment of $M^1 = Ae^{-J_1\Phi_k}\sin K_1\Phi_k + Be^{-J_1\Phi_k}\cos K_1\Phi_k + \ldots$ is induced.

By symmetry the effect of a moment at the left-hand edge would be similar. Combining the effects from the two edges

$$M_o = Ae^{-J_1\Phi_k}\sin K_1\Phi_k + B(1 + e^{-J_1\Phi_k}\cos K_1\Phi_k) + Ce^{-K_1\Phi_k}\sin J_1\Phi_k + D(1 + e^{-K_1\Phi_k}\cos J_1\Phi_k) \; (38)$$

or $M_o = Aa_1 + Bb_1 + Cc_1 + Dd_1 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (39)$

where a_1, b_1, c_1, d_1 are constants depending on the shell dimensions.

Similarly we can express N_2, T_2, $\dfrac{dS}{dx}$, etc., in the form

$$\left.\begin{aligned} N_2 &= Q_{N_2}(Aa_2 + Bb_2 + Cc_2 + Dd_2) \\ T_2 &= Q_{T_2}(Aa_3 + Bb_3 + Cc_3 + Dd_3) \\ \frac{dS}{dx} &= \\ T_1 &= \\ v &= \\ w &= \\ \psi &= Q_\psi(Aa_8 + Bb_8 + Cc_8 + Dd_8) \end{aligned}\right\} \quad . \quad . \quad . \quad (40)$$

(The quantities M_2 T_2 T_1 and w are symmetrical about the crown and the effect of the forces at left and right sides are added, whereas the quantities N_2, $\dfrac{dS}{dx}$, v and ψ are antisymmetrical and for symmetrical edge loads and displacements the effects from the left-hand side are to be subtracted from those of the right-hand side.)

Consider now an edge load of $N_2 = \dfrac{4}{\pi}$, $T_2 = \dfrac{dS}{dx} = \psi = 0$.

Then

$$Q_{N_2}(Aa_2 + Bb_2 + Cc_2 + Dd_2) = \frac{4}{\pi}$$

$$Q_{T_2}(Aa_3 + Bb_3 + Cc_3 + Dd_3) = 0$$

$$Q_S(Aa_4 + Bb_4 + Cc_4 + Dd_4) = 0$$

$$Q_\psi(Aa_8 + Bb_8 + Cc_8 + Dd_8) = 0$$

These can be solved for A, B, C and D for any value of $K_1\Phi_k$ (and thus $J_1\Phi_k$). Since Q_{N_2} varies a solution in the form

$$A = \frac{A^1}{Q_{N_2}} \qquad B = \frac{B^1}{Q_{N_2}} \qquad C = \frac{C^1}{Q_{N_2}} \qquad D = \frac{D^1}{Q_{N_2}}$$

can be obtained.

Substituting these values back in the expressions (32) and (33) the edge values of T_1, v, w due to unit load N_2 are found. They are of the form :

$$v_{N_2} = \frac{Qv}{Q_{N_2}}\left\{A^1a_6+B^1b_6+C^1c_6+D^1d_6\right\}$$

Similarly the forces and displacements due to other loading cases can be found.

The expression in brackets is tabulated for the displacements and longitudinal stress in Table I, the ratio of the factors Q being the term at the head of the appropriate columns.

At any point θ in the shell, measured from the right-hand edge, we have

$$M = Ae^{-J_1\theta} \sin K_1\theta + \ldots .$$

and from the left-hand edge the angle is so that

$$M^1 = Ae^{-J_1(\Phi_k-\theta)} \sin K_1(\Phi_k - \theta) + \ldots$$

Thus the total value of M is

$$M\theta = A(e^{-J_1\theta} \sin K_1\theta + e^{-J_1(\Phi^k-\theta)} \sin K\,(\Phi_k - \theta) + \ldots$$

Due to a unit load N_2 putting

$$A = \frac{A^1}{Q_{N_2}} \quad B = \frac{B^1}{Q_{N_2}} \text{ etc., as found above}$$

$$M\theta_2\,(N_2) = \frac{1}{Q_{N_2}}\left(A^1a^1{}_1+B^1b^1{}_1+C^1c^1{}_1+D^1d^1{}_1\right)$$

where a^1, b^1, etc., are constants depending on the shell dimensions and the angle θ.

and similarly we can obtain expressions in the form

$$T_2\theta(N_2) = \frac{Q_{T_2}}{Q_{N_2}}\left\{A^1a^1{}_3 + B^1b^1{}_3 + C^1c^1{}_3 + D^1d^1{}_3\right\}$$

for the various forces.

The terms in parentheses are tabulated in Tables IIa-IIe for the values of $\theta = 0, \frac{\Phi_k}{8}, \frac{\Phi_k}{4}, \frac{3\Phi_k}{8}$ and $\frac{\Phi_k}{2}$

The values of the displacements at the intermediate points in the shell are not given as these are generally of no interest.

APPENDIX III

In some papers, e.g. (9), it has been stated that the Schorer approximation leads to considerable errors.

It is the opinion of the author that this has yet to be proved and that all available evidence points to the conclusion that for all "practical" structures the errors are insignificant with the exception of the class of shells classified as "short shells" where the radius and chord width are much greater than the span. Those interested in the comparison of the various methods should refer to the paper by Dr. J. Macnamee[8] and the ensuing discussion.

Table IV shows a comparison of the shell forces for the barrel vault roof taken in the numerical example worked out

(a) on a slide rule (results taken from this paper).

(b) on a machine using Schorer approximation.

(c) on a machine using design method based on Jenkins equation.

		Edge	$\frac{1}{8}$ Point	$\frac{1}{4}$ Point	$\frac{3}{8}$ Point	Crown
T_1	a	− 4860	− 10,200	− 12 300	− 12 230	− 12 070
	b	− 5 755	− 10,650	− 12 205	− 12 035	− 11,755
	c	− 6 160	− 10,650	− 12 200	− 11,960	− 11,700
S	a	+ 8,360	+ 6 860	+ 4660	+ 2 265	0
	b	+ 8,420	+ 6,800	+ 4,580	+ 2 260	0
	c	+ 8400	+ 6 760	+ 4,550	+ 2 250	0
T_2	a	− 120	− 755	− 1,241	− 1,531	− 1,626
	b	− 121	− 762	− 1,239	− 1,522	− 1616
	c	− 123	760	− 1,232	− 1,513	− 1,605
N_2	a	+ 37 0	− 57 4	− 79 4	− 51 6	0
	b	+ 39 0	− 54 7	− 76 8	− 50·4	0
	c	+ 41·8	− 51 1	− 73 8	− 48 5	0
M_2	a	+ 316	+ 255	− 5 1	− 259	− 360
	b	+ 314	+ 260	− 3 5	− 250	− 349
	c	+ 269	+ 233	− 2·4	− 226	− 314

TABLE IV (ALL FORCES IN LB FT UNITS)

If the results by the more exact methods are compared with those obtained here it will be seen that where the only error of any size occurs, i.e., the edge value of T_1, the effect on the reinforcement is negligible.

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Discussion

The Literature Committee would be glad to consider the publication of correspondence in connection with the above paper. Communications on this subject intended for publication should be received by September 1st, 1954.

Institution Notices and Proceedings

ORDINARY GENERAL MEETING

An Ordinary General Meeting of the Institution of Structural Engineers was held at 11, Upper Belgrave Street, London, S.W.1, on Thursday, April 22nd, 1954, at 5.55 p.m., Lt.-Colonel R. F. Galbraith, M.C., B.Sc., M.I.Struct.E., A.M.I.C.E. (President), in the Chair.

The following members were elected in accordance with the Bye-Laws. Will members kindly note that the elections as tabulated below should be referred to when consulting the Year Book for evidence of membership.

STUDENTS

CANESSA, Eric Albert, of Loughborough, Leicester.
CHUNG CHENG CHEK, of Singapore.
DOYLE, Thomas Hewett, of Benoni, Transvaal, South Africa.
ELLIS, Ralph John, of Broxbourne, Herts.
GLOWINSKI, Tadeusz Zygmunt, of London.
ILOTT, Derek Thomas, of Reading, Berks.
McCARTHY, Patrick O'Gorman, of London.
McCORMACK, Alan Henry, of Manchester.
McGLADDERY, Alan Wilson, of Hayes, Middlesex.
NICHOLAS, Wilfred Arasaratnam, of Maradana, Ceylon.
VAGGERS, F/O, George Graham, of Newquay, Cornwall.
VAN ESSEN, Albert, of Bloemfontein, South Africa.

GRADUATES

BARTLETT, Leslie Norman Edward, of Sunbury-on-Thames, Middlesex.
BENTON, Roy, of London.
BOWLES, Anthony John Hastings, B.Sc.(Eng.) London, of Kingston-upon-Thames, Surrey.
BROWN, Arthur Sydney, of Carshalton, Surrey.
BUTTERWORTH, James Robert, of Manchester.
CLAYDEN, Eric John, of London.
DRIMER, Morris, B.Sc.(Eng.) London, of Leeds, Yorks.
DUTTA-MUNSI, Tushar Kanti, B.E.(Civil) Calcutta, of London.
FOK WAI KUEN, of Hong Kong.
FRANKS, Stanley Fitzgerald, of London.
FREHE, John, A.R.I.B.A., B.Arch. Durham, of Onchan, Isle of Man.
GENEROWICZ, Bohdan Sedimir, B.Sc.(Eng.) London, of London.
GHOSH, Subimal, B.E. Calcutta, of London.
HOBIN, Richard Edward, B.Arch. Auckland, of London.
LAHIRI, Supravat Chandra, B.E.(Civil) Calcutta, of Newmains, Lanarkshire.
LEA, William Nigel, B.Sc.(Civil) Birmingham, of London.
LOEDOLFF, Pieter Botha, B.Sc.(Civil) Cape Town, of Salisbury, Southern Rhodesia.
MORRIS, Gordon Ronald, of Southsea, Hants.
NEWEY, Gerald, of Liverpool.
PAI, G. Ramachandra, B.E.(Civil) Madras, of Madras, India.
PEACH, George Derrick, of North Wembley, Middlesex.
PICKNETT, John Kay, B.Sc.(Eng.) London, D.I.C., of London.
RAHULAN, Govindan, of Singapore.
RENDLE, Philip John, B.Sc.(Eng.) London, of Plymouth.
RIDEN, Alfred Donald, of Liverpool.
SANVILLE, Stephen Colin, M.A.(Cantab.), of Wilmslow, Cheshire.
SEYMOUR, Bryant Abel, of Abertridwr, Glamorgan.
SMITH, Michael John Chambers, A.M.I.C.E., A.M.I.Mun.E., of Durham City.
STEAN, John George, of Rochester, Kent.
STEWART, John, B.Sc.(Civil) Glasgow, of Glasgow.
TAIWO, Emanuel Ola, of London.
TAYLOR, Roy, B.Sc.(Eng.) Nottingham, of Hatfield, Herts.
TONNER, Eric William, of Auckland, New Zealand.
WEATHERLEY, Noel, of Birmingham.
WEIDEMA, Piet, of London.
WELLER, Alan David, of Watford, Herts.
WEPENER, Adolf Casimir, A.M.I.C.E., of Johannesburg, South Africa.
WILLIAMS, Kenneth Wilfred, of Bebington, Cheshire.
WONG SAU TUEN, of Hong Kong.

ASSOCIATE-MEMBERS

ELLIOTT, John Cameron, B.Sc.(Eng.) London, of London.
LAW, Geoffrey Thomas, A.M.I.C.E., of Littleover, Derby.
McKAY, Bernard John, of London.
SMITH, Robert Bernard Louis, B.Sc.(Tech.) Manchester, of Chinley, Stockport, Cheshire.
SPOONER, Leslie Allan, of Nairobi, Kenya.
WILLIAMS, Thomas Eifion Hopkins, M.Sc., Ph.D., F.S.S., of Whitley Bay, Northumberland.
WOOLDRIDGE, Harold Albert, of London.

MEMBER

ROBERTS, Gilbert, B.Sc., M.I.C.E., of London.

TRANSFERS

Students to Graduates

ARCHER, John Bartley, of London.
BAGNALL, John Burfitt, of Bromley, Kent.
BARWIS, Edward Charles, of London.
BILLINGTON, Roy, of Huyton, nr. Liverpool. Lancs.
BINGHAM, David Malcolm, of Liverpool.
BONTOFT, Anthony John, of Scunthorpe, Lincs.
BOWLEY, Robert, of Tring, Herts.
BROWN, Francis Stanley, of Scunthorpe, Lincs.
CLARK, John Alan, of London.
COX, Bruce Albert, of London.
DODD, James Michael, of Ellesmere Port, Wirral, Cheshire.
DOWELL, David Keith, of Bromborough, Cheshire.
FINNIS, Roy, of London.
FLETCHER, Kenneth Albert, of Bexleyheath, Kent.
FRANCIS, Rhys Hugh, of London.
HALL, Edward Tuffnell, of Walsall, Staffs.
HARTLE, George, of Rochdale, Lancs.
HUNT, Henry William, of Thornton Heath, Surrey.
McHUGH, Patrick Thomas, of Twickenham, Middlesex.
MATTOCKS, Ronald, of London.
ODEDAIRO, Ebenezer Olufunso, of Leicester.
PEARSON, Matthew Thomas, of Hebburn-on-Tyne, County Durham.
SUMMERBELL, George Bernard, of Newcastle upon Tyne.
VENIER, John, of London.
WEDGE, Henry Robert, of London.

Graduates to Associate-Members

ASTILL, Alan Walter, B.Sc.(Eng.), of Sutton Coldfield, Warwicks.
BIRD, Brian Cecil, B.Sc.(Eng.) London, of Ambergate, Derbyshire.
BLOW, Leslie William Furse, of London.
BRIDGE, Stuart Berry, of Glossop, Derbyshire.
BROTTON, Derick Maxwell, B.Sc.(Eng.) London, Ph.D., of Middlesbrough, Yorks.

BUCHBINDER, Michael, of Haifa, Israel.
CHAPMAN, William Edwin, A.M.I.C.E., of Hope, nr. Sheffield.
CROWDEN, Brian Bertram, of Birmingham.
DOWSETT, Victor Frederick, of London.
DUNN, William Nicoll, of London.
GREEN, Reginald William, of Stretford, Manchester.
GREENHALGH, Fred, of Bolton, Lancs.
HUTTER, James Louis, of Watford, Herts.
KEY, David Edwin, B.Sc.(Eng.) London, of London.
LAU FOO SUN, B.Sc.(Eng.) London, D.I.C., of Ferrybridge, Yorks.
LEVELL, Donald John, B.Sc.(Eng.) London, A.M.I.Mun.E., of Southampton.
NEEDHAM, Frederick Harold, B.Sc.(Eng.), London, A.C.G.I., of West Wickham, Kent.
NEWBY, Frank, B.A.(Cantab.), of London.
PARSONS, Geoffrey Frank, of London.
PATRICK, John George, B.Sc.(Eng.) London, A.C.G.I., of Banstead, Surrey.
PLANT, Gordon Vickers, of Eccles, nr. Manchester.
SENIOR, Alan Gordon, M.Sc. Leeds, B.Sc.Hons., of St. Albans, Herts.
SMEDLEY, Alan, of Salford, Lancs.
STONEBRIDGE, Marcus Allan, A.M.I.C.E., A.M.I.Mun.E., of Bath, Somerset.
STRINGER, Robert George Alexander, A.M.I.C.E., A.M.I.Mun.E., of Kingston upon Hull, Yorks.
TAYLOR, John Godfrey, of Bolton, Lancs.
WALTON, Frank Thompson, D.L.C.(Hons.), B.Sc.(Eng.) London, A.M.I.C.E., of London.
WHITTAKER, Dennis Beatty, B.Sc.(Tech.) Manchester, A.M.I.C.E., of Whitehaven, Cumberland.

Associate-Members to Members

ANTIA, Khursed Framroz, B.Sc.(Eng.) London, of Bombay, India.
CORNISH, Ronald James, M.Sc., M.I.C.E., A.M.I.Mech.E., of Manchester.
GRIFFITHS, Leslie John, of Worcester.
WRIGHT, George, of Hale Barns, nr. Altrincham, Cheshire

Associate-Members to Retired Associate-Members

BARTON, Andrew Raynor, A.M.I.Mech.E., of Cambridge.
ROBINSON, Harold William, of North Harrow, Middlesex.

Members to Retired Members

COFF, Leo, of New York, U.S.A.
VERNALL, Richard John, O.B.E., V.R.D., A.R.I.B.A., (Retd.) of Bromyard, Herefordshire.

Re-Admission to Membership (from Retired Membership)

KNIGHT, Bernard Howard, D.Sc. Ph.D. London, M.I.C.E., F.R.I.C.S., of Wraysbury, Berks.

OBITUARY

The Council regret to announce the death of JOHN PERCY CLARK (Member), HAROLD WORRALL (Associate-Member), HECTOR ST. GEORGE ROBINSON (Retired Member), and Flying-Officer MAURICE SIDNEY SPINKS (Graduate).

JUNE MEETING

An Ordinary General Meeting of the Institution, for the election of members only, will be held at 11, Upper Belgrave Street, London, S.W.1, on Thursday, June 24th, 1954, at 5 p.m.

EXAMINATIONS—JANUARY, 1954

OVERSEAS CENTRES

The examinations were held in January, 1954, overseas, at the following centres :—

Aden.
Aligarh.
Bombay.
Bulawayo.
Brisbane.
Calcutta.
Canberra.
Cape Town.
Chittagong.
Colombo.
Cooma.
Durban.
East London (South Africa)
Enugu.
Hong Kong.
Jerusalem.
Johannesburg.
Karachi.
Kuala Lumpur.
Lahore.
Lucknow.
Madras.
Melbourne.
Miri (Sarawak).
Montreal.
Port Elizabeth.
Salisbury (S. Rhodesia).
San Francisco.
Singapore.
Sydney.
Tel Aviv.
Toronto.
Tripoli (Lebanon).
Wellington.

Thirteen candidates took the Graduateship Examination, and eighty-one candidates took the Associate-Membership Examination, making a total of ninety-four. Of these, four passed the Graduateship Examination, and nineteen passed the Associate-Membership Examination.

The names of the successful candidates are :—

GRADUATESHIP EXAMINATION

ALLMAN, Laurence Mills.
CHOWDHARY, Abdul Wahid.
D'SYLVA, Eunan Declan.
SAXENA, Kailash Chandra.

ASSOCIATE-MEMBERSHIP EXAMINATION

BOWMAN, Charles William.
CHEN TSING-KWAN.
CHITALE, Vaman Mahadeo.
DAVAR, Kersi S.
DONALD, Allan.
KRUGER, Manuel Mandel.
KULKARNI, Shantaram Vishnu.
LEKSHMANAN, K. N.
LYNCH, William Henry.
MALLOWS, Dennis Lingwood.
MITRA, Asit Kumar.
MULGREW, Raymond Maurice.
NOLLER, Gerald Rous Allen
RAMAKRISHNA, Hanasoge Suryanarayana A.
SALMON, Norman Derek.
STANKIEWICZ-WISNIEWSKI, Kazimierz.
TAN CHIN THYE.
VAN GYSEN, Theodorus Johannes.
WARDLE, Terence Michael.

PRIZES—JANUARY, 1954 EXAMINATIONS

The Council have awarded the following prizes, in respect of the examinations held in January, 1954.

ANDREWS PRIZE. (For the candidate who obtains the highest aggregate of marks in the Associate-Membership Examination, passing in all subjects.)

Gerald Rous Allen NOLLER, of Wellington, N.Z.

HUSBAND PRIZE. (For the candidate who takes the whole of the Associate-Membership Examination, passes in all subjects, and obtains the highest marks in the paper, "Structural Engineering Design and Drawing.")

Gerald Rous Allen NOLLER, of Wellington, N.Z.

WALLACE PREMIUM (SENIOR). (For the candidate who takes the whole of the Associate-Membership Examination, passes in all subjects, and obtains the highest marks in the paper, "Theory of Structures (Adv.)".)

Gerald Rous Allen NOLLER, of Wellington, N.Z.

WALLACE PREMIUM (JUNIOR). (For the most successful candidate in the Graduateship Examination, passing in all subjects.)

Arthur Sydney BROWN, of Carshalton, Surrey.

EXAMINATIONS—JULY, 1954

The Examinations of the Institution will next be held at centres in the United Kingdom and overseas on July 13th and 14th, 1954 (Graduateship), and July 15th and 16th (Associate-Membership).

CORRECTION

It is greatly regretted that owing to a clerical error it was stated on page 133 of the April issue of the Journal that "Dr. A. J. Ockleston had been appointed Principal of the University of Witwatersrand." We would ask

both Professor W. G. Sutton, who is, of course, the Principal of the University of the Witwatersrand, and Professor A. J. Ockleston, of the Department of Civil Engineering of the University of the Witwatersrand, to accept our most sincere apology for any inconvenience and embarrassment which this error may have caused them.

SOUTH AFRICAN INSTITUTION OF CIVIL ENGINEERS

The South African Institution of Civil Engineers have decided to accept the Associate-Membership Examination of the Institution of Structural Engineers, if passed after the Royal Charter was granted, as a qualification for corporate membership.

REPRESENTATION

The Council have appointed Mr. Walter C. Andrews (Past President) and Mr. Gower B. R. Pimm (Past President) as the Institution's Representatives on the new Codes of Practice Council which has been set up within the British Standards Institution.

OVERSEAS REPRESENTATION

Mr. K. G. Stevens (Member), the Institution's Representative in Southern Rhodesia, has been appointed by the Council as the Institution's Representative for the Federation of Northern Rhodesia, Nyasaland and Southern Rhodesia.

WITHDRAWAL OF PUBLICATION

The Council have resolved to withdraw Forms of Contract "A" and "B," but would draw the attention of members to the Model Form "C" issued by the Association of Consulting Engineers.

CRICKET MATCH

The Annual Cricket Match between an Institution of Structural Engineers XI and The Blue Circle Club will be played on Saturday, July 10th, 1954, at the Blue Circle Club's ground, Bromley, Kent, starting at 11.30 a.m.

Cricketers in any grade of membership of the Institution who would like to play in this game should get in touch with Mr. D. A. G. Reid, L.C.C. Brixton School of Building, Ferndale Road, S.W.4, as soon as possible.

RESEARCH AWARDS

The Council has instituted a Research Prize Fund, from which awards may be made annually to the author or joint authors of papers describing original research which they have carried out. Research awards may be made for papers read at Headquarters or in the Branches and published in the Journal, or for papers published in the Journal only without being read at an open meeting.

The assessment for such awards will be made annually, but awards will be made only to the contributors of such papers as reach a standard judged by the Literature Committee to be satisfactory.

Work submitted under this scheme must be original and may include any of the following :—

(a) investigations of an experimental or analytical character ;
(b) studies of historical or statistical records ;
(c) improvements in principles or methods of construction ;
(d) research into methods of structural engineering and building, the nature and use of plant and the organisation of engineering work ;
(e) any related or combined studies which are deemed by the Literature Committee to be of a research character.

In cases where the research work described in the paper was not the work of one individual, the names of all the collaborators should be given in the paper.

Awards may take any or all of the following forms :— A research medal ; a diploma ; a money prize.

Application for consideration for a research award must be made to the Secretary of the Institution, and in preparing papers for reproduction in the Journal, authors must comply with the conditions laid down for all such contributions. Particulars of these conditions may be obtained from the Secretary.

In judging research papers, the following factors will be considered :—

(a) the nature of the subject and its conclusions ;
(b) the value of the paper in advancing the science and art of structural engineering ;
(c) the standard of preparation and orderly arrangement of the subject-matter.

Research papers will also be eligible for adjudication for the Institution Medals if they comply with the Regulations governing those awards.

The closing date for the receipt of applications in respect of papers published in the Journal between October, 1953, and September, 1954, is October 1st, 1954.

YEAR BOOK AND LIST OF MEMBERS

The Year Book and List of Members for 1954 will go to press in July, for publication in October, when a copy will be sent to all members.

Members are requested to inform the Secretary of any alterations in titles, degrees or addresses, which have not already been notified, by July 5th, in order that such amendments may be included in the new edition.

INSTITUTION LIBRARY

During the past four years, the Institution Library has remained open until 6 p.m. from Monday to Friday during the Session. So little use has been made of this facility, however, that the Council have decided that the arrangement will be discontinued at the end of the current Session. From the 30th June, therefore, the Library will be closed at 5 p.m., except on evenings when Ordinary Meetings of the Institution are held, when it will remain open until 6 p.m.

EXAMINATIONS

Preparation for the Examinations of the Institution by Attendance at Technical Colleges

A candidate for Graduateship or Associate-Membership may be able to attend a technical college ; these notes are intended to guide him in choosing the most suitable instruction.

Preparation for the Graduateship Examination

Technical Colleges offer :

(a) Full-time courses for degrees or Higher National Diplomas in Building or Engineering.

(b) Part-time day or evening courses for Higher National Certificates in Building or Engineering.

If he obtains a Higher National Certificate or Diploma complying with Appendix II, Section V, of the Regulations Governing Admission to Membership, the candidate will be exempted from the Graduateship Examination.

Alternatively, he may study subjects selected from the available courses and sit the Graduateship Examination. At technical colleges courses are usually available in Building Science or Engineering Science, Strength of Materials, Theory of Structures and Surveying, but students are not normally allowed to select subjects from National Diploma or Certificate courses unless they can show evidence of sound training in more elementary studies. The advice of the College Authorities should be followed.

Preparation for the Associate-Membership Examination

At some technical colleges there are part-time courses in Structural Engineering which cover the syllabus of the Associate-Membership Examination. At other colleges the candidate must rely on Higher National Certificate courses or on advanced courses in Building, Civil Engineering or Municipal Engineering; these cover only part of the requirements for the Associate-Membership Examination.

Colleges in List "A" provide at least two years of instruction in Theory of Structures and in Structural Engineering Design and Drawing up to Associate-Membership standard. They also give instruction in Structural Specifications, Quantities and Estimates.

List "A"

- Bath Technical College.
- Belfast College of Technology.
- Birmingham College of Technology.
- Bolton Municipal Technical College.
- Bradford Technical College.
- Bridgend Technical College.
- Chesterfield College of Technology.
- Derby Technical College.
- Dudley and Staffordshire Technical College.
- Glasgow Royal Technical College.
- City of Liverpool College of Building.
- L.C.C. Brixton School of Building, S.W.4.
- L.C.C. Hammersmith School of Building and Arts and Crafts, W.12.
- Manchester College of Technology.
- Middlesbrough, Constantine Technical College.
- Nottingham and District Technical College.
- Salford, Royal Technical College.
- South-East London Technical College, Lewisham Way, S.E.4.
- South-West Essex Technical College, Walthamstow, E.17.
- Stockport College for Further Education.
- Twickenham Technical College.
- Willesden Technical College, N.W.10.

Colleges in List "B" provide instruction in Theory of Structures from which the student may reach Associate-Membership standard, but instruction in Structural Engineering Design and Drawing and in Structural Specifications, Quantities and Estimates is not usually so complete.

List "B"

- Brighton Technical College.
- Cardiff Technical College.
- Edinburgh, Heriot-Watt College.
- Huddersfield Technical College.
- Leeds College of Technology.
- London, Battersea Polytechnic, S.W.11.
- London, Northampton Polytechnic, E.C.1.
- L.C.C. Westminster Technical College, S.W.1.
- Newcastle upon Tyne, Rutherford College of Technology.
- Plymouth and Devonport Technical College.
- Preston, Harris Institute.
- Rotherham College of Technology.
- Wigan Mining and Technical College.
- Woolwich Polytechnic, S.E.18.

Students are advised to take the organised courses in Structural Engineering where these are available.

LONDON GRADUATES' AND STUDENTS' SECTION

A visit to the steelworks of Messrs. Dawnays Limited at Welwyn Garden City has been arranged for the morning of Saturday, July 10th. Lunch will be provided; return fare from Kings Cross, 5/6d.

The number of visitors is limited to 35 and those wishing to participate should make early application to the Hon. Secretary, J. A. Pope, 53, Cranleigh Drive, Leigh-on-Sea, Essex.

BRANCH NOTICES

LANCASHIRE AND CHESHIRE BRANCH

Joint Hon. Secretaries: A. S. Sinclair, A.M.I.Struct.E., 17, The Circuit, Cheadle Hulme, Cheshire; M. D. Woods, A.M.I.Struct.E., 58, Spring Gardens, Salford, 6.

MIDLAND COUNTIES BRANCH

Hon. Secretary: L. A. Firminger, A.M.I.Struct.E., 656, Chester Road, Erdington, Birmingham, 23.

GRADUATES' AND STUDENTS' SECTION

Hon. Secretary: H. L. Bramwell, 139, Wood Lane, Handsworth Wood, Birmingham, 20.

NORTHERN COUNTIES BRANCH

Hon. Secretary: Captain O. Lithgow, A.M.I.Struct.E., 4, Stoneleigh Avenue, Acklam, Middlesbrough.

NORTHERN IRELAND BRANCH

Hon. Secretary: A. H. K. Roberts, B.A., B.A.I., M.I.Struct.E., M.I.C.E.I., "Barbizon," 26, Dunlambert Park, Belfast.

SCOTTISH BRANCH

Hon. Secretary: G. Drysdale, A.M.I.Struct.E., "Niaroo," 33, Union Street, Motherwell, Lanarkshire.

SOUTH-WESTERN COUNTIES BRANCH

Joint Hon. Secretaries: E. W. Howells, M.I.Struct.E., 10-12, Market Street, Torquay, Devon; C. J. Woodrow, "Elstow," Hartley Park Villas, Tavistock Road, Plymouth, Devon.

WALES AND MONMOUTHSHIRE BRANCH

A Junior Members' Evening was held at Swansea on February 17th, when three papers were given by students. These papers were judged by a Sub-Committee for prizes given by the Branch but the standard of the papers was such that it was recommended that three prizes be awarded.

Prizes have therefore been awarded as follows:—

First. Mr. N. L. Longley (University student), for a paper on "Prestressed Concrete."

Second. Mr. J. D. Brunt (Student, Institution of Structural Engineers), for a paper entitled "An Outline of Steel Fabrication."

Third. Mr. J. M. Lomax (University student), for a paper on "Methods of Coast Protection."

Hon. Secretary: G. R. Brueton, A.M.I.C.E., A.M.I.Struct.E., 86, The Exchange, Mount Stuart Square, Cardiff.

WESTERN COUNTIES BRANCH

Hon. Secretary: E. Hughes, A.M.I.Struct.E., 23, Southdown Road, Westbury-on-Trym, Bristol, 9.

YORKSHIRE BRANCH

Hon. Secretary: E. Wrigley, A.M.I.Struct.E., 17, The Drive, Alwoodley, Leeds.

UNION OF SOUTH AFRICA BRANCH

Branch Hon. Secretary: A. E. Tait, B.Sc., A.M.I.C.E., A.M.I.Struct.E., P.O. Box 3306, Johannesburg. During week-days Mr. Tait can be contacted in the City Engineer's Department, Town Hall, Johannesburg. 'Phone: 34-1111, Ext. 257.

Natal Section Hon. Secretary: E. G. Bennett, A.M.I.Struct.E., c/o The Reinforcing Steel Co., Ltd., P.O. Box 49, Merebank, Durban.

Cape Section Hon. Secretary: R. Stubbs, M.I.Struct.E., The Reinforcing Steel Co., Ltd., P.O. Box 2962, Cape Town.

The Failure Load of Rigid Jointed Frameworks as Influenced by Stability

By W. Merchant, M.A., S.M., A.M.I.Struct.E.

Synopsis

Stanchions have been used for a long time as component members of rigid framed structures. At present they are designed by analogy with the behaviour of isolated stanchions. This method is open to criticism and obscures the action of a stanchion as a part of a structure. This paper advances arguments that the structure itself instead of its component parts be designed by analogy with an isolated stanchion.

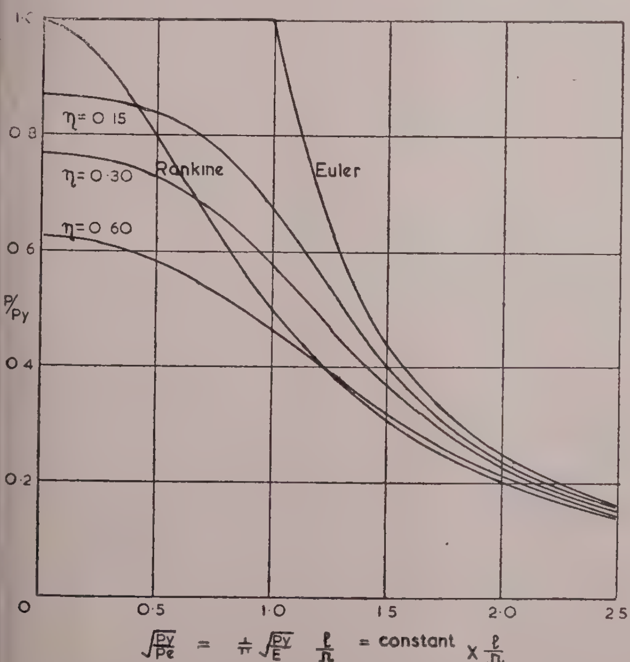
Introduction

The behaviour of a pin ended axially loaded stanchion in a testing machine is well understood.

Stanchions are however used as component parts of building and other structural frames and here the position is not so satisfactory. The natural growth of design methods has led to comparing the behaviour of stanchions in structures with that of isolated stanchions. For example, in building codes of design one may be allowed to assume that the effective length (that is, the length of the stanchion between points of zero bending moment, i.e., the equivalent pin ended stanchion) is a constant fraction, say, 0.7 of the storey height due to the stiffness of the beam to stanchion connections. These methods have not led to anything more than very empirical methods of design of unknown accuracy. The reason is that as loads increase the effective lengths alter as the stiffness of members is dependent on the axial load they carry and is not constant. A different approach is required and one is suggested in this paper.

Isolated Stanchions

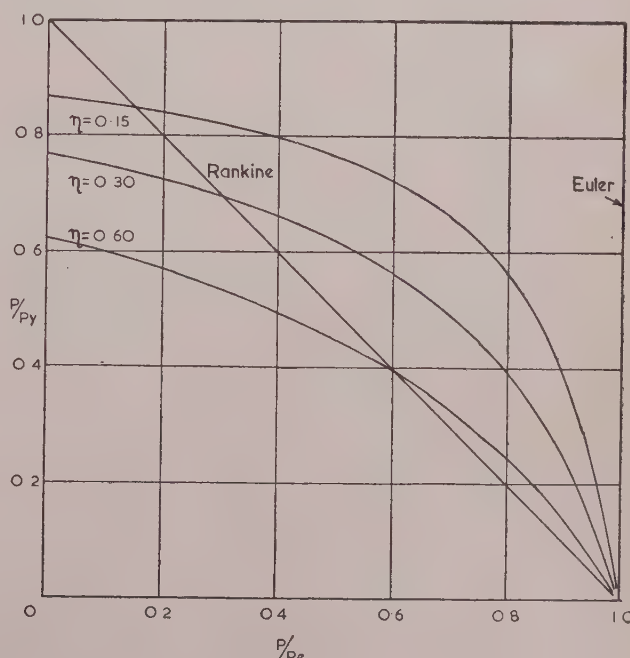
At an early stage in the history of structural engineering, the stability of an elastic pin ended stanchion was examined and an expression was derived by Euler for the critical load, that is the load at which the stanchion would cease to offer any resistance to disturbing forces or moments and accidental imperfections would be magnified without limit if the material remained elastic. The analysis is too well-known to repeat here ; it yields an infinite number of critical loads but it is only the lowest which is of interest to structural engineers. Below this load the straight form of the strut is stable against small disturbances, above it the straight form is unstable. The Euler load of a stanchion then only depends on the elastic properties and not on the strength of the material of which the member is made. If a stanchion is long and slender its failure load is in fact



For rectangular sections $\eta = 3\frac{\Delta_0}{d}$ where $2d$ is the section width.

The constant in the expression for η varies with the shape of the section.

Fig. 1a



Perry-Robertson Strut Curves

Fig. 1b

very nearly given by the Euler load, but for short members the yield stress of the material is of paramount importance. For stanchions of intermediate length both the stress corresponding to the Euler load and also the yield stress have to be taken into consideration. At the present time the art has developed to such an extent that we can write :

$p_f = f(p_e, p_y, \eta)$ where . . . . (1)

p_f = mean stress corresponding to the failure load.
p_e = mean stress corresponding to the Euler load.
p_y = yield stress.
η = an arbitrary parameter representing imperfections of manufacture and of testing conditions.

Sufficient is known about the form of the function to have a reasonably complete representation of the behaviour of pin ended stanchions in testing machines.

For example, Fig. 1 shows the well-known Perry-Robertson curves

$(2p_f = (p_y + (1+\eta)p_e) - \sqrt{(p_y + (1+\eta)p_e)^2 - 4p_yp_e})$

plotted non-dimensionally so that they apply to materials in general whatever their yield point. In Fig. 1a they are plotted so as to resemble ordinary strut curves, whilst a more symmetrical type of plotting representing the same information is shown in Fig. 1b. For interest, the curve for the Rankine Formula, an earlier but nevertheless intelligent type of "engineering interpolation," has also been included.

The parameter which determines whether stability effects are important is $\dfrac{p_y}{p_e}$. If p_y is large compared with p_e then the failure will be almost completely determined by stability considerations, and $\dfrac{p_f}{p_e}$ will be nearly equal to unity since accidental imperfections do not affect critical loads.

If p_y is small compared with p_e then stability effects are negligible. If there are accidental imperfections there will be bending moments on the stanchion as well as axial loads. The Perry-Robertson Formula allows for these by limiting the maximum stress to the yield stress. This accounts for $\dfrac{p_f}{p_y}$ being less than unity for small values of $\dfrac{p_y}{p_e}$ when η has a finite value.

Complete Structures

The new approach suggested for complete structures is derived by analogy from the isolated stanchion case and is as follows :—

We suggest that for a particular loading pattern on a structure

$P_f = f(P_c, P_y, \eta)$ where . . . . (2)

P_f = the actual failure or collapse load of the structure if all the loading is increased slowly and in constant proportion.
P_c = the critical load of the structure, i.e., the load at which it would offer no resistance to applied disturbances if its members remained elastic.
P_y = the collapse load of the structure if only material properties are taken into account and stability effects ignored, i.e,. limit load.
η = a parameter corresponding to the one representing imperfections for the isolated strut and about which more will be said later.

Let us consider each of the terms on the right-hand side of equation (2) in turn.

Consideration of P_c

Consider the portal shown in Fig. 2.

The vertical members AB and CD carry axial loads and also have bending moments acting on them. The behaviour of members under both axial loads and bending was first investigated during the 1914-18 War. Modified forms of the Theorem of Three Moments and Slope Deflection Equations were obtained using Berry

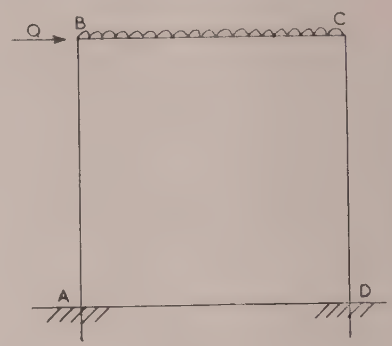


Fig. 2

Functions. Accounts of the early methods and tables of Berry Functions are to be found in References (1) and (2).

Since that time the tendency has been for successive approximation methods (Moment Distribution, Relaxation, etc.), to be used in place of formal mathematical methods. These methods can also be used for dealing with members under combined axial loads and bending if the concepts of stiffness and carry-over factors are introduced. The small diagram on Fig. 3 shows the moments at the ends of a member under axial loading. s is the non-dimensional stiffness factor ; its value is 4 if stability effects are negligible, c is the non-dimensional carry-over factor, its value is $\frac{1}{2}$ if stability effects are negligible. s and c are clearly related to the Berry Functions and they are functions of $\dfrac{P}{P_E}$. s and c have not been much used in literature in this country and so graphs of their values are given in Fig. 3 ; they are derived from data in reference (3).

For problems about critical loads or limit loads it is convenient to think of a loading parameter. If we keep the pattern of loads on a structure constant and increase all the loads in constant proportion, then the loading parameter is the measure of the magnitude of the loads.

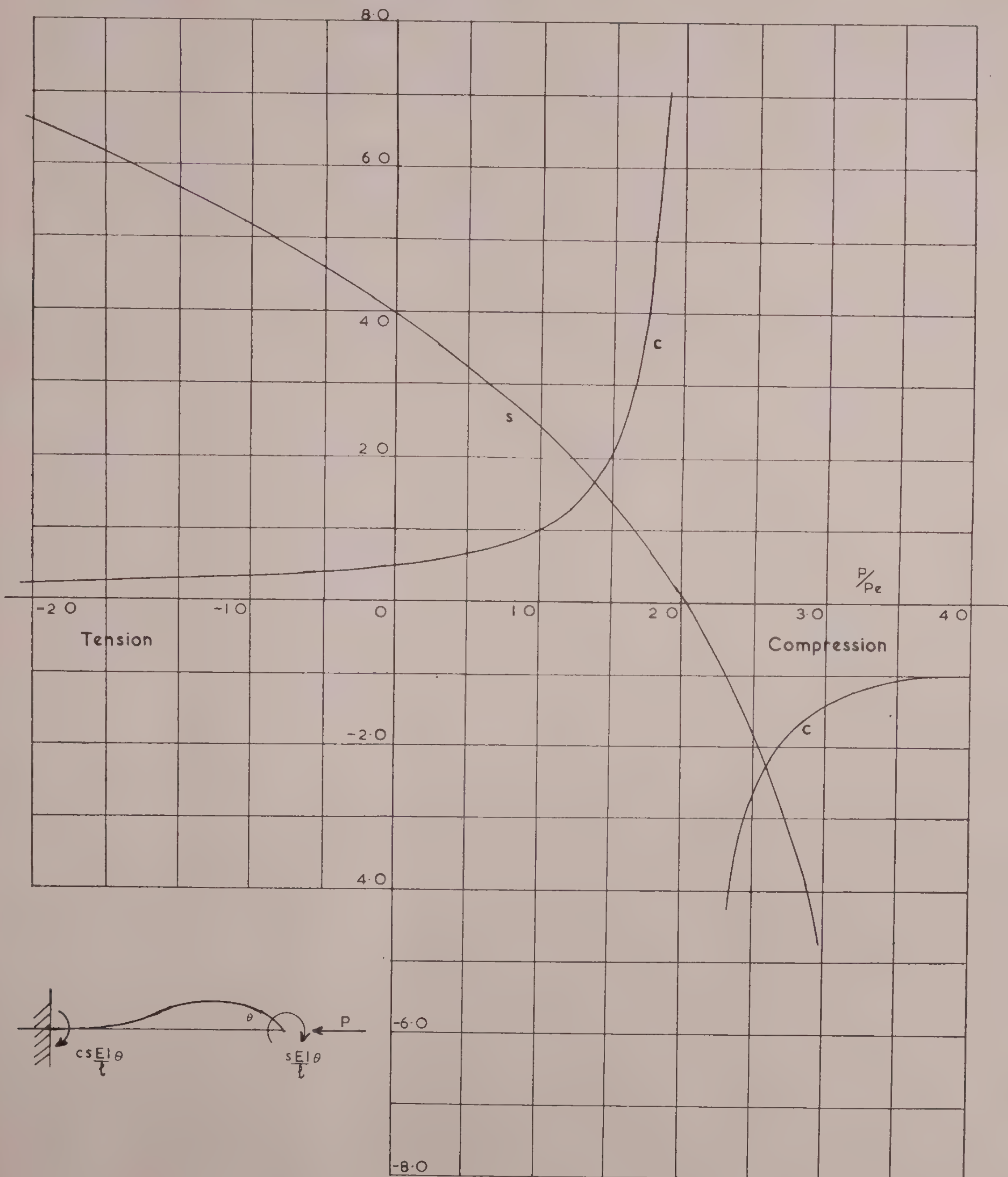
For any particular value of the loading parameter for the portal in Fig. 2 we can by an ordinary moment distribution calculation determine the stiffness of the portal to an applied small disturbing force such as Q.

Let the deflection corresponding to Q be $\triangle$. We define the stiffness K of the complete structure to the particular disturbing force Q by the relationship $Q = K\triangle$. Thus, if the stiffness is small the deflections are large and conversely. If we carry out calculations for different values of the loading parameter P we shall be able to plot a graph of K against P, as is shown in Fig. 4. P_c is given by the intercept on the P axis. P_c corresponds in all respects to the Euler load for an isolated stanchion. At P_c the structure as a whole has no resistance to disturbances and no matter how strong the material of which it is composed it will be unusable.

For complicated structures the calculations may have to be done numerically. Unfortunately, near the critical load the convergence of the moment distribution calculations is slow. The process may be speeded by

an extrapolation technique due to Lundquist[6] and in other ways but much work has still to be done in improving the methods of calculating the critical loads of complex structures. A simple example which does not need calculation of individual points is given in Appendix I to illustrate the type of curve obtained in practice. References 1, 4 and 5 may be consulted for further data about critical loads.

The critical load for the complete structure is what is important in determining the behaviour of the structure and the $\left(\frac{P}{P_E}\right)$ values for the individual members are only indirectly of importance in so far as they determine the stiffness and carry over factors of the members. Depending on the structure considered, the $\left(\frac{P}{P_E}\right)$ values of the individual members at the critical load of the complete structure may vary widely and may be greater or less than unity.



Stiffness and Carry Over Factors for Members with Axial Loads

Fig. 3

Consideration of P_y

So far, much more progress has been made in determining P_y than P_c, and as this work will be familiar to structural engineers it will not be discussed further here. (References 7, 8, 9, 10.) We will note that whilst P_y for an isolated stanchion is determined by yielding under axial loads, for a rigid frame structure on the other hand,

the effects of bending moments will be of paramount importance.

CONSIDERATION OF η

Let us first consider an analysis of an isolated pin ended stanchion due to Southwell[11].

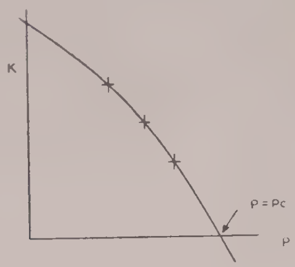


Fig. 4

The differential equation of equilibrium of a strut not initially straight is

$$P_y + EI\left(\frac{d^2y}{dx^2} - \frac{d^2y_o}{dx^2}\right) = 0 \quad \ldots\ldots \quad (3)$$

Where y_o is the original shape as shown in Fig. 5.

Fig. 5

Equation (3) can be solved in Fourier Series.

Assume $y_o = \Sigma\, a_n \sin \frac{n\pi x}{l}$

$y = \Sigma\, b_n \sin \frac{n\pi x}{l}$

Then substituting in equation (3) we have

$$\frac{P}{EI}\Sigma b_n \sin \frac{n\pi x}{l} - \frac{\pi^2}{l^2}\Sigma n^2 (b_n - a_n) \sin \frac{n\pi x}{l} = 0$$

and this must be true for all values of x ($0 < x < l$).

Therefore

$$\frac{Pl^2}{\pi^2 EI} b_n - n^2 (b_n - a_n) = 0$$

$$\text{i.e., } b_n = \frac{a_n}{1 - \frac{P}{EI}\frac{l^2}{n^2\pi^2}} \quad \ldots\ldots \quad (4)$$

Hence

$$y = \Sigma \frac{a_n \sin \frac{n\pi x}{l}}{l - \frac{P}{EI}\frac{l^2}{n^2\pi^2}} \quad \ldots\ldots \quad (5)$$

Now y is large whenever $P = \frac{n^2\pi^2 EI}{l^2}$

These values of P are the critical loads and there are an infinite number P_{c_1} P_{c_2}, etc. The first $P_{c_1} = \frac{\pi^2 EI}{l^2}$ and so on.

$$\text{Hence } y = \frac{a_1 \sin \frac{\pi x}{l}}{l - \frac{P}{P_{c_1}}} + \frac{a_2 \sin \frac{2\pi x}{l}}{l - \frac{P}{P_{c_2}}} + \ldots \quad (6)$$

Near any critical load Pc_n the term corresponding to it predominates and we can write

$$y \approx \frac{a_n \sin \frac{n\pi x}{l}}{1 - \frac{P}{Pc_n}}$$

There is therefore a characteristic mode of buckling associated with each critical load, that of the corresponding term of the Fourier expansion. The mode associated with Pc_1 is a single sine curve, the mode associated with Pc_2 is a double sine curve, and so on. This explains why even if the original shape of the strut is far from sinusoidal, its component in the shape of a sine curve will be magnified much more than its other components and will predominate at the first critical load. We can if we like think of y_o as being expressed in terms of the buckling modes instead of as terms in a Fourier expansion, and this is a more fundamental concept.

The η values for an isolated stanchion are non-dimensional measures of the magnitude of the component of the imperfection of shape of the stanchion in the first buckling mode. Stanchions with different original imperfections of shape or different imperfections of testing conditions will have the same failure load if they have the same values of η.

We propose to take over these concepts from an isolated stanchion to a complete structure and to think of the buckling modes of the structure corresponding to its critical loads. Let the displacements in the buckling modes from the centre lines of the undistorted structure be y_1, y_2, etc. Then we will think of any deflected shape of the structure as being analysed in terms of these modes.

i.e., $y = a_1y_1 + a_2y_2 + a_3y_3 + \ldots\ldots$

If the deflected shape of the structure when stability effects can be ignored is

$y_o = a_1y_1 + a_2y_2 + a_3y_3 + \ldots\ldots$

then exactly as for an isolated strut we propose that

when stability effects have to be taken into account the deflected shape will be given by

$$y = \frac{a_1 y_1}{1 - \dfrac{P}{Pc_1}} + \frac{a_2 y_2}{1 - \dfrac{P}{Pc_2}} + \ldots \quad (7)$$

We do not know of any general proof in mathematical terms of equation (7) although we have proved it for certain special cases. However, Southwell's method of deducing critical loads from experimental observations which is derived from this analysis has been used for structures in general and not only for isolated stanchions and Lundquist's method of extrapolating stiffness curves also depends on the generalisation. There appears therefore to be good warrant for accepting equation (7) even though a formal proof is at present lacking.

The first deduction we will make from equation (7) is that small distortions of shape of rigid structures, whether they arise from manufacture or from loading, will not affect the values of the critical loads. This is also the case for isolated stanchions.

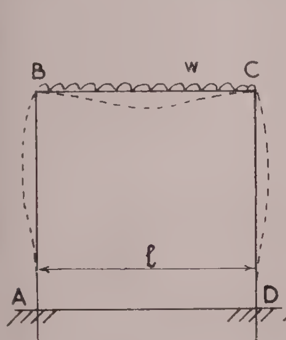


Fig. 6a

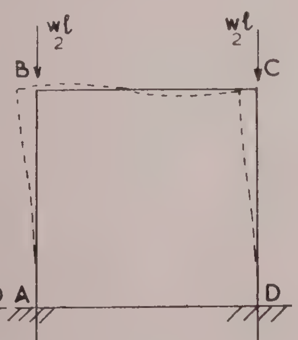


Fig. 6b

Thus the two portals shown in Fig. 6 will have almost exactly the same critical loads as they have the same axial forces in the stanchions. They will only differ in that the portal on the left has an axial force in member BC whilst the portal on the right has no axial force. In building frames where the axial forces in the beams are usually small compared with those in the stanchions this will have a negligible effect. Critical loads in building frames will in nearly all cases be determinable by treating the loads as lumped at the panel points and there will be no need to treat rigorously the harder problems of the effect of distributed loading.

Our second deduction from equation (7) is that the value of η for a complete structure is concerned with the component of the deflected shape of the structure in the first buckling mode. This is again in complete analogy with the case of the isolated stanchion.

The lowest critical mode of the portal shown in Fig. 6 is associated with side sway and is shown dotted in Fig. 6b. The deflected shape of the portal under symmetrical loading indicated in Fig. 6a contains no anti-symmetrical terms and hence no component of the first buckling mode. The η value for symmetrical loading will therefore be very small and we can expect the failure loads of the portals in Figs. 6a and 6b to be very nearly the same despite the original deflection of the portal with the distributed loading.

On the other hand, if sway is prevented then the lowest buckling mode of the portal is approximately as shown in Fig. 6a. In this case, although both portals will have the same critical load the one on the left will have a much higher η value than the one on the right and we can expect its failure load to be reduced accordingly.

A third comment is that for rigid frame structures loaded away from panel points the limit loads are largely determined by bending moments and the effect of any small imperfections of manufacture will be small. Thus we can expect to represent failure conditions in rigid frame structures on a diagram as given in Fig. 7 where all the curves will converge at $\dfrac{P_y}{P_c} = 0$ as well as at $\dfrac{P_c}{P_y} = 0$.

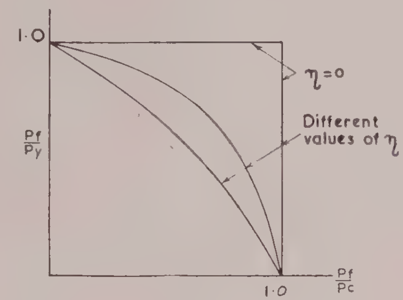


Fig. 7

It is too early yet to define the various curves accurately but even at the present time we could put the design of complete structures on a more logical base by working to some such safe curve as the analogue of the Rankine curve for struts, i.e.,

$$\frac{P_f}{P_c} + \frac{P_f}{P_y} = l$$

Such a course would go a long way to clear up some of the anomalies of present design methods.

Conclusions

A new method has been suggested for determining the failure loads of structures where stability effects are important.

The method is still in its early stages and further work is required.

So far work has been started on the experimental determination of such curves as are sketched in Fig. 7 and on theoretical methods of calculating critical loads of complex structure.

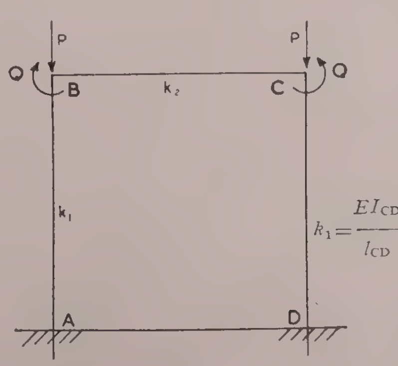


Fig. 8

Acknowledgement

This paper describes the background to a research project being undertaken in the Department of Building

and Structural Engineering, College of Technology, Manchester.

Appendix

Investigate the structure shown in Fig. 8 for a disturbing force of two equal and opposite moments Q applied to the joints at B and C.

The moments at the joints given below are tabulated in the manner used by Bolton[12].

	A		B		C	D
Rotation B (1) ...	csk_1	sk_1	$4k_2$	$2k_2$	0	0
Rotation C (2) ...	0	0	$2k_2$	$4k_2$	sk_1	csk_1
(1) — (2) = (3)	csk_1	sk_1	$2k_2$	$-2k_2$	$-sk_1$	$-csk_1$

Line 3 gives the moments in the portal for equal and opposite unit rotations at B and C.

The total moment at $B = (sk_1+2k_2) = Q$
and hence $K = sk_1+2k_2$

When $P = 0$, $S = 4$ and $\dfrac{K}{K_o} = \dfrac{s + \dfrac{2k_2}{k_1}}{4+2\dfrac{k_2}{k_1}}$ and a curve similar to that in Fig. 4 can now be plotted directly.

NOTE : The calculation above determines the lowest critical load of the portal if sway is prevented.

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An Experimental Investigation of the Behaviour of Mild Steel Compression Members in Light Lattice Frameworks

By S. Mackey, M.E., B.Sc., Ph.D., A.M.I.C.E., A.M.I.Struct.E.

Introduction

Design of light lattice girders and trusses composed of structural steel angles is commonly based on the recommendations of B.S.S.449-1948. Tension members forming such girders are designed to comply with clauses 39 and 40 of the specification and experimental research[1] has indicated that members proportioned in accordance with these clauses give calculated yield loads which agree closely with the measured loads.

Clauses 17, 18, 19 and 21 of the specification refer to the design of single angle compression members. The mechanics of failure of such members involves several factors, which depend upon the truss as a whole, the individual member concerned, its adjacent members and its end connections. Theoretical concepts upon which the design of compression members is based must be supported by ample experimental evidence involving destructive testing of frames before a satisfactory design procedure can be achieved. Most of the current strut formula have been confirmed by tests on single members without allowing for the effects due to rotation and displacement of the end connections which form an integral part of frame action.

The object of the investigation described in this paper was to determine the critical loads and stresses causing failure of compression members in plane triangulated frames and to consider the difference between ultimate loads and those causing first yielding of the members.

Scope of Tests

Two complete girders were first tested to failure, the results of which are given elsewhere.[2,3] These were followed by tests on a series of mild steel elemental frames similar to those shown in Fig. 1 and Plate 1.

Eleven different frames were tested, four of these being examined under fully-bolted and singly-bolted end conditions. Details of the angles forming the frames are given in Table I. In this table, cross-sectional areas based on measured angle thicknesses are given together with the percentage variation from the nominal area in each case. Structural properties for all angles tested, are also recorded.

Typical stress distributions in the members under load are shown in Fig. 2. These are derived from readings on E.R.S. gauges affixed to the members as indicated in

Fig. 1. Fig. 3 shows the type of variation in bending moment which occurred along the length of the members under different test loads with reference to bending about the U-U and V-V axes. The M_u and M_v moments were resolved into M_x moments causing bending in the plane of the girder. At low loads the signs of these M_x moments agreed with the calculated secondary moments

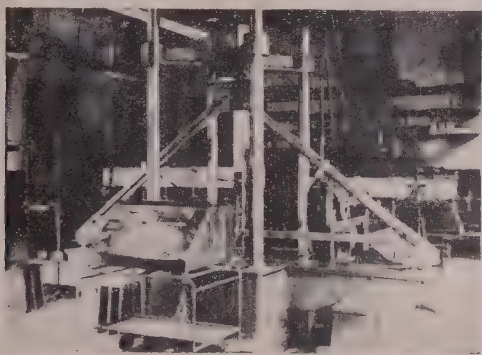

Plate 1

due to joint rigidity but as buckling proceeded their effect was completely subdued. It may be inferred therefore that secondary moments causing bending in double-curvature have little effect on the primary buckling mode. In Fig. 3 the dotted lines indicate estimated values of the moments when yielding begins and the method of computing moments by elastic theory breaks down.

Deflection records of the centroid at the mid-length of the member were obtained for each angle tested. The movements involved are indicated on Fig. 4 for a representative number of cases. Load-deflection graphs are plotted for deflections parallel to the principal axes. For convenience of reference these are referred to as lateral deflections (δ_L) and bending deflections (δ_B). (See Fig. 4.)

In compiling Table II, values of the applied load eccentricities E_B and E_L have been computed about the principal axes on the assumption that the load acts at the mid-thickness of the attached leg on the gauge line for the bolts. The corresponding experimental eccentricities e_B and e_L were obtained by dividing the recorded moments M_u and M_v by the applied load in the member, thus giving an effective moment arm. The difference between this moment arm and the value of the corresponding at the mid-length of the member was taken as the effective eccentricity e_B. In no case did the value of e_B alter appreciably as the frame load was increased. The values given in Table II are therefore applicable to all loadings within the elastic limit for the frames.

Effective lateral eccentricity e_L tended to alter with alteration in the frame load. The values obtained were higher for the heavy angles than for those of lighter section, but at no time during the tests did they reach serious proportions. In general it was found that allowance for effective lateral eccentricity e_L is sufficiently well covered in the Perry formula by an allowance of

$$0.003 \frac{l}{r} \text{ for the parameter } \left(\frac{ae}{r^2}\right).$$

Plots of the effective eccentricities on the angle cross-sections showed that the load application points varied considerably. It is clear, however, that for fully-bolted angles the present ruling of B.S.S. No. 449 regarding assumed loading eccentricities meets the cases examined. In the case of the angles with single bolts the current allowance of the full eccentricity of loading in the plane of the gusset appears unduly conservative from the results obtained. This, however, may be due to an unduly high frictional restraint built up as a result of the large gusset plates used.

In no instance was ultimate load reached until yielding had developed over most of the angle cross-section, as shown on Fig. 5. The central moment increased with increasing load while the end moments first increased, then decreased, and eventually changed sign as the joint restraints were developed. Yielding first occurred at

TABLE I

Test Mark	Angle Size	No. of Tests S.B. †	No. of Tests F.B. ‡	Connected Leg (in.)	Back Mark (in.)	Bolt Diam. (in.)	Measured Area (in.²)	Variation from Nominal Area (%)	Elastic Limit tons/in.²	Yield Stress tons/in.²	Ultimate Stress tons/in.²	Young's Modulus tons/in.²
1	$4 \times 3\frac{1}{2} \times \frac{1}{2}$*		1	4	$2\frac{1}{4}$	$\frac{3}{4}$	3.48	—0.58	16.30	19.80	30.20	13,000
2	$3\frac{1}{2} \times 3 \times \frac{1}{2}$*		1	$3\frac{1}{2}$	2	,,	2.87	—0.43	12.80	15.60	28.40	13,200
3	$3\frac{1}{2} \times 2\frac{1}{2} \times \frac{3}{8}$		1	$3\frac{1}{2}$	2	,,	2.08	—1.42	14.90	17.50	30.80	13,000
4	$3 \times 2 \times \frac{3}{8}$		1	3	$1\frac{3}{4}$	,,	1.70	—1.73	15.10	17.60	30.00	13,400
5	$2\frac{1}{2} \times 2 \times 5/16$		1	$2\frac{1}{2}$	$1\frac{3}{8}$	,,	1.35	+3.06	14.20	17.40	29.60	13,190
6	$2\frac{1}{2} \times 1\frac{1}{2} \times \frac{1}{4}$		2	$2\frac{1}{2}$	$1\frac{3}{8}$	,,	0.91	—2.87	14.60	16.86	27.40	13,000
7	$2 \times 1\frac{1}{2} \times \frac{1}{4}$		1	2	$1\frac{1}{8}$	$\frac{5}{8}$	0.85	+4.70	14.80	17.40	28.40	13,100
8	$3 \times 3 \times \frac{1}{2}$		1	3	$1\frac{3}{4}$	$\frac{3}{4}$	2.56	—6.90	14.20	17.40	30.90	13,400
9	$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{3}{8}$		1	$2\frac{1}{2}$	$1\frac{3}{8}$	,,	1.66	—4.05	14.70	17.70	29.50	13,190
10	$2\frac{1}{4} \times 2\frac{1}{4} \times 5/16$		2	$2\frac{1}{4}$	$1\frac{1}{4}$	$\frac{5}{8}$	1.25	—4.56	13.20	16.10	25.40	13,000
11	$2 \times 2 \times \frac{1}{4}$		1	2	$1\frac{1}{8}$	,,	0.94	0	15.90	18.45	28.40	13,100
12	$2\frac{1}{4} \times 2\frac{1}{4} \times 5/16$	1		$2\frac{1}{4}$	$1\frac{1}{4}$	$\frac{5}{8}$	1.25	—4.56	13.20	16.10	25.40	13,000
13	$2 \times 2 \times \frac{1}{4}$	1		2	$1\frac{1}{8}$	,,	0.94	0	15.90	18.45	28.40	13,100
14	$2\frac{1}{2} \times 1\frac{1}{2} \times \frac{1}{4}$	1		$2\frac{1}{2}$	$1\frac{3}{8}$	$\frac{3}{4}$	0.91	—2.87	14.60	16.86	27.40	13,000
15	$2 \times 1\frac{1}{2} \times \frac{1}{4}$	1		2	$1\frac{1}{8}$	$\frac{5}{8}$	0.85	+4.70	14.80	17.40	28.40	13,100
16	$3\frac{1}{2} \times 2\frac{1}{2} \times \frac{3}{8}$		1	$2\frac{1}{2}$	$1\frac{3}{8}$	$\frac{3}{4}$	2.08	—1.42	14.90	17.50	30.80	13,000
17	$2\frac{1}{2} \times 2 \times 5/16$		1	2	$1\frac{1}{8}$	$\frac{5}{8}$	1.35	+3.06	14.20	17.40	29.60	13,190

*Denotes Frame B.
†S.B. denotes Singly Bolted.
‡F.B. denotes Fully Bolted.

the mid-section (with symmetrical buckling). After this had developed, the load could still be increased until the moments at the end reached their maximum value, which caused either yield in the end sections, rotation of the joint or yield of the gusset in bending.

From consideration of the stress-distribution diagrams over the cross-section of the members, values of the ratio $\frac{\text{max. stress}}{\text{mean stress}}$ have been obtained. These are shown in Table III and apply within the elastic range. The ratio is generally greater for unequal angles, than for equal in Table III. Included also in this table are results obtained by Stang and Strickenberg[1] from an investigation on structural steel angles carried out by them.

In compiling the slenderness ratios observed effective lengths obtained from the deflection profiles have been used. In view of the relatively high value obtained with the $2\frac{1}{2}$ in. $\times$ $1\frac{1}{2}$ in. $\times$ $\frac{1}{4}$ in. angle a duplicate test was carried out and an almost identical result obtained.

Fig. 6, which has been derived from the results of Table III, compares the experimental results with current design formulæ. Curve (2), which represents the Perry formula of B.S.S. 449-1948 with a straight-line relation-

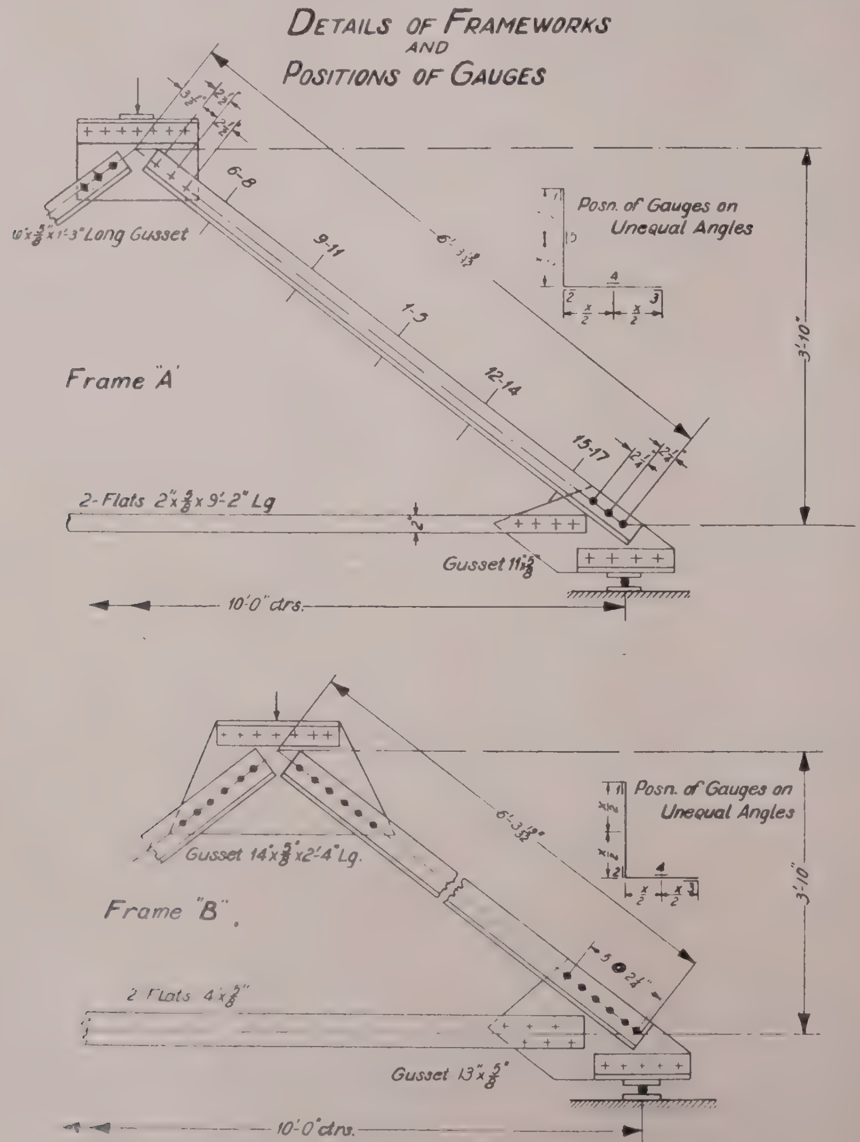

Fig. 1

angles, the average values obtained being 2.48 and 2.38 respectively for fully-bolted conditions with corresponding values of 3.0 and 3.7 for single-bolt fastenings. It appears therefore that for slender members requiring single-bolts at the ends, unequal angles with the short leg outstanding are preferable to equal angles, whereas for heavier members requiring more than one bolt the reverse is true.

Critical Loads and Stresses

For purposes of comparison with existing design methods the collected results of the strut tests are given

ship between $\frac{l}{r} = 80$ and $\frac{l}{r} = 0$ is plotted for a yield stress of 17.40 tons in.^{2} representing the average yield of the tensile specimens. Curve (3) corresponds to Table VIII of the same specification using a value of 0.785 for the parameter $\frac{ac}{r^2}$ and adopting the yield point as the critical stress. The curve is drawn for critical stress and not for permissible stress. The values plotted

for the test results obtained by Stang and Strickenberg denote the stress causing first yield in the extreme fibres of the strut. These values have been deduced by application of Fig. 7 to the critical buckling stresses recorded by the original investigators. In Fig. 7 the ratio $\frac{\text{max. load}}{\text{yield load}}$ is plotted against slenderness ratio for both equal and unequal angles. This ratio increases with decreasing slenderness ratio and is greater for

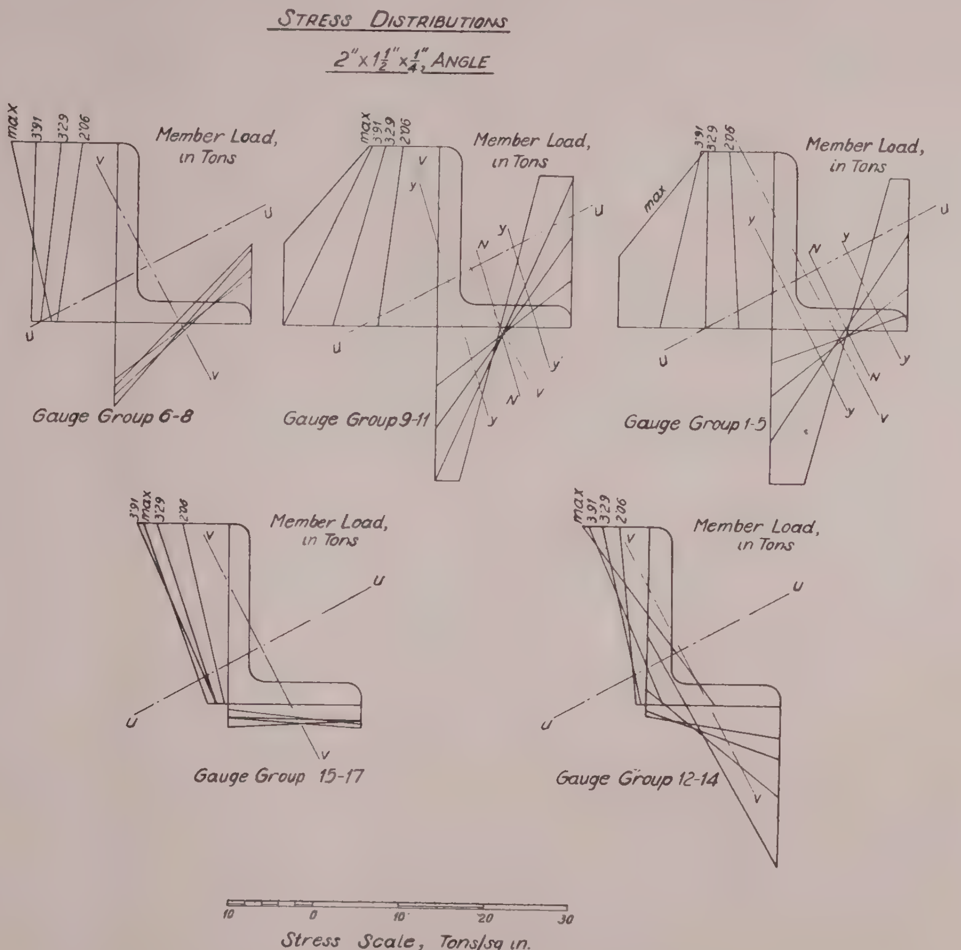

Fig. 2

TABLE II

Test Mark	Angle Size	E_B (in.)	E_L (in.)	e_B (in.)	e_L (in.)	$\frac{e_B}{E_B}$	Measured Load (Tons)	$\frac{\text{Max Stress}}{\text{Mean Stress}}$	Effective Length (in.)	$\frac{l}{L}$
1	4×3½×½	1.25	—0.1	1.32	+0.17	1.06	27.1	2.39	48	0.80
2	3½×3×½	1.07	0	0.98	+0.15	0.91	16.5	2.11	48	0.80
3	3½×2½×⅜	0.98	+0.04	0.57	+0.28	0.58	12.3	2.63	50	0.74
4	3×2×⅜	0.78	+0.06	0.58	—0.06	0.75	11.9	2.53	49	0.73
5	2½×2×5/16	0.70	+0.03	0.37	—0.23	0.53	8.2	2.77	48	0.72
6	2½×1½×¼	0.57	+0.06	0.82	—0.08	1.44	4.9	2.43	46	0.71
7	2×1½×¼	0.55	+0.03	0.34	—0.03	0.62	3.9	2.74	45	0.67
8	3×3×½	1.06	—0.11	0.90	+0.03	0.85	10.7	2.95	67.5	1.00
9	2½×2½×⅜	0.84	—0.04	0.64	+0.06	0.76	12.4	2.21	48	0.72
10	2¼×2¼×5/16	0.77	—0.05	0.48	+0.10	0.62	7.4	1.88	45	0.67
11	2×2×¼	0.71	—0.02	0.28	+0.01	0.39	5.8	2.48	44	0.65
12	2¼×2¼×5/16	0.77	—0.05	0.62	+0.04	0.81	4.1	3.80	56	0.83
13	2×2×¼	0.71	—0.02	0.65	+0.02	0.91	3.7	3.85	67.5	1.00
14	2½×1½×¼	0.57	+0.06	0.43	—0.10	0.76	4.9	2.04	48	0.71
15	2×1½×¼	0.55	+0.03	0.33	+0.07	0.60	4.3	3.02	49	0.73
16	3½×2½×⅜	1.18	—0.23	1.08	—0.04	0.92	10.0	2.67	54	0.80
17	2½×2×5/16	0.85	—0.15	0.67	—0.06	0.79	4.1	3.34	48	0.72

equal angles than for unequal angles. Bleich[4] states that for short columns and certain sections the difference between the load causing first yield and that producing buckling may reach values ranging from 30 per cent. to 40 per cent. of the buckling load. In the present number of members tested in this condition is not sufficient to determine this conclusively. Since it is unlikely that singly-bolted angle struts will be widely used within this range of slenderness ratio only one is submitted which lies almost directly on Curve (3).

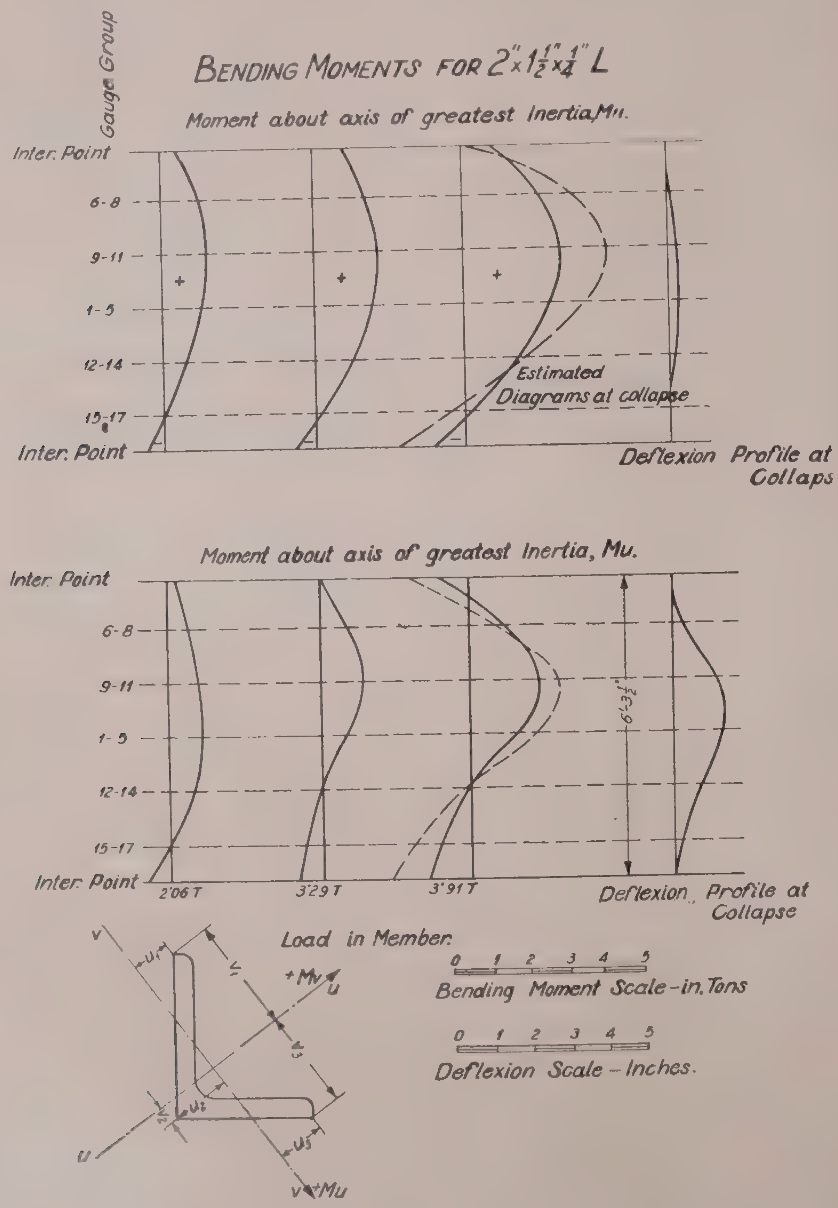

Fig. 3

investigation differences of this magnitude were not reached but the results bear out the validity of the statement in principle.

Consideration of the results shown in Table III and Fig. 6 leads to the following conclusions :—

1. For slenderness ratios above 145 the results obtained for singly-bolted and fully-bolted struts, in the present investigation and in that carried out by Stang and Strickenberg, lie well above Curve (3) and are in fact close to the Euler curve.

2. For slenderness ratios ranging from 90 to 145 the experimental results for fully-bolted struts with the short leg outstanding lie between the Perry Curve and Curve (3). Those for fully-bolted struts with the long leg outstanding appear to follow Curve (3) but the

3. Below slenderness ratios of 80 the present writer's results lie close to Curve (3) whereas the mean value of Stang and Strickenberg is approximately mid-way between this curve and the modified Perry curve.

Calculated Critical Stress Values

The calculated critical stress values in Table III have been obtained from the formula :—

$$f_a = \frac{F_p}{\left\{ 1 + \dfrac{F_p\ f_a}{F_y^2} \left(\dfrac{a_1\ c_B}{r^2_{max}} \right)^2 \right\}}$$

(See Appendix A for Nomenclature).

Test Mark	Angle Sizes	$\frac{l}{r_{\min}}$	$\frac{a_1 e_B}{r_{\max}^2}$	Measured Stresses: Yield fcr_1	Measured Stresses: Buckling fcr_2	Ratio $\frac{fcr_2}{fcr_1}$	Euler Fe	Computed Stresses: Perry F_p	Computed Stresses: B.S.449 F_2	fa	Ratios: $\frac{fcr_1}{Fe}$	Ratios: $\frac{fcr_1}{F_p}$	Ratios: $\frac{fcr_1}{F_2}$	Ratios: $\frac{fcr_1}{fa}$
1	$4\times3\frac{1}{2}\times\frac{1}{2}$	69.5	1.82	8.28	9.43	1.14	—	12.02	8.00	6.49	—	0.69	1.03	1.28
2	$3\frac{1}{2}\times3\times\frac{1}{2}$	78.6	1.58	7.32	8.75	1.19	—	11.33	7.50	6.90	—	0.64	0.98	1.06
3	$3\frac{1}{2}\times2\frac{1}{2}\times\frac{3}{8}$	94.5	0.96	7.31	8.13	1.11	—	9.47	6.66	7.75	—	0.77	1.10	0.94
4	$3\times2\times\frac{3}{8}$	117	0.55	7.02	7.53	1.07	—	7.10	5.40	6.55	—	0.99	1.30	1.07
5	$2\frac{1}{2}\times2\times5/16$	115	0.87	7.19	8.37	1.16	—	7.28	5.51	6.50	—	0.99	1.31	1.10
6	$2\frac{1}{2}\times1\frac{1}{2}\times\frac{1}{4}$	148	2.06	5.88	6.52	1.11	5.88	4.86	3.98	3.95	1.00	1.21	1.48	1.49
7	$2\times1\frac{1}{2}\times\frac{1}{4}$	145	1.03	4.65	4.65	1.00	6.10	5.01	4.09	4.70	0.77	0.93	1.14	0.99
8	$3\times3\times\frac{1}{2}$	116	1.53	5.94	7.37	1.24	—	7.19	5.45	5.41	—	0.83	1.09	1.10
9	$2\frac{1}{2}\times2\frac{1}{2}\times\frac{3}{8}$	100	1.26	7.71	9.21	1.20	—	8.82	6.31	6.73	—	0.87	1.22	1.15
10	$2\frac{1}{4}\times2\frac{1}{4}\times5/16$	105	1.05	7.52	8.64	1.15	—	8.30	6.02	6.90	—	0.91	1.25	1.09
11	$2\times2\times\frac{1}{4}$	113	0.69	7.00	7.80	1.14	—	7.47	5.61	6.90	—	0.94	1.25	1.01
12	$2\frac{1}{4}\times2\frac{1}{4}\times5/16$	130	1.38	4.70	5.59	1.19	—	6.02	4.74	5.04	—	0.78	0.99	0.93
13	$2\times2\times\frac{1}{4}$	173	1.62	4.03	4.20	1.04	4.31	3.69	3.15	3.30	0.94	1.09	1.28	1.22
14	$2\frac{1}{4}\times1\frac{1}{2}\times\frac{1}{4}$	150.5	1.08	5.67	5.67	1.00	5.68	4.72	3.89	4.35	1.00	1.20	1.46	1.30
15	$2\times1\frac{1}{2}\times\frac{1}{4}$	158	0.99	5.26	5.26	1.00	5.20	4.33	3.62	4.25	1.01	1.21	1.45	1.24
16	$3\frac{1}{2}\times2\frac{1}{2}\times\frac{3}{8}$	102	1.42	6.45	8.42	1.31	—	8.60	6.20	6.32	—	0.75	1.04	1.02
17	$2\frac{1}{2}\times2\times5/16$	115	1.30	4.70	5.67	1.21	—	7.38	5.54	5.95	—	0.64	0.85	0.79
	Av. of 5	71.2		9.52	11.20									
Double Bolts	Av. of 9	113		5.80	6.30									
	Av. of 14	150		5.36	5.47									
	Av. of 7	192		3.75	3.75									
	Av. of 4	182		4.30	4.30									
Single Bolts	Av. of 10	228		2.44	2.44									
	Av. of 17	273		1.89	1.89									
	Av. of 6	318		1.30	1.30									

Results by Stang and Strickenberg.

Denotes that allowance has been made in $\frac{l}{r}$ ratio for End Fixity.

Values obtained from corresponding buckling stresses by application of Fig. 7.

LATERAL DEFLEXIONS OF CENTROIDS WITH INCREASING LOAD

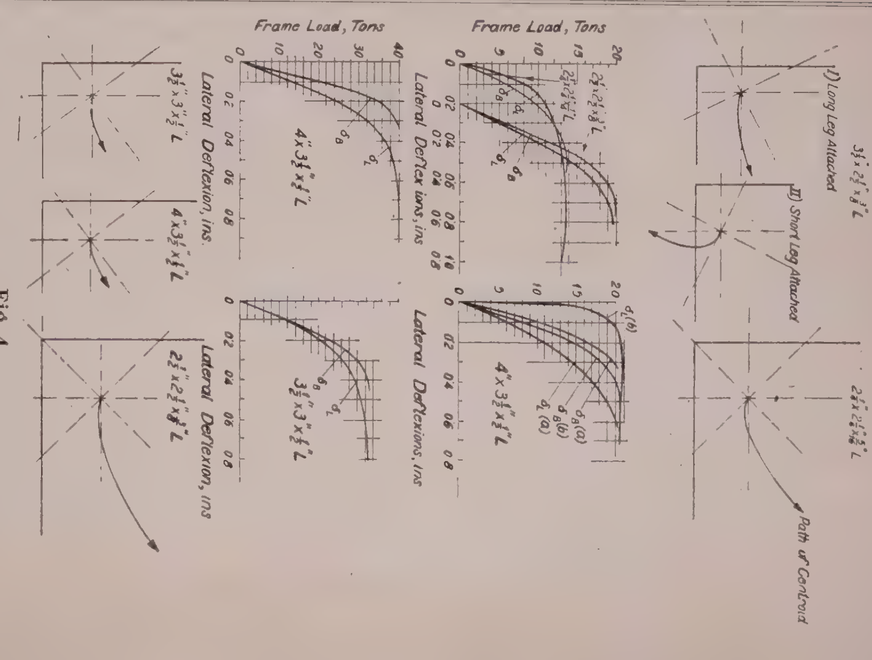

Fig. 4

DIAGRAMS SHOWING SPREAD OF YIELD ZONE AT LOADS NEAR BUCKLING LOAD

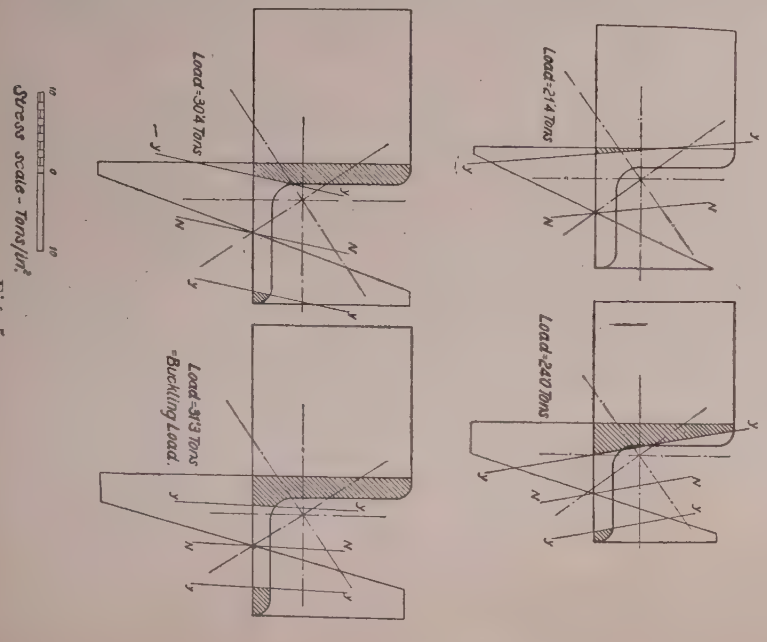

Fig. 5

Examination of the results shows that for the fully-bolted struts closer agreement with the experimental data is obtained using this formula than by applying the method adopted in B.S.S. 449-1948. Only in three of the fully-bolted tests is the calculated load greater than the observed load. One of these refers to an angle strut with long leg outstanding which is inherently weaker than an identical angle strut connected through the longer leg and is therefore to be avoided in practical design. In the second case the difference is only 1 per cent. of the actual load and may therefore be neglected. In the remaining instance no obvious reason can be given for the discrepancy which, however, does not exceed 7 per cent. of the observed load.

As an indication of the accuracy achieved in carrying out the experimental work duplicate tests were carried out on the $2\frac{1}{4}$ in. × $2\frac{1}{4}$ in. × 5/16 in. angles and on the $2\frac{1}{2}$ in. × $1\frac{1}{2}$ in. × $\frac{1}{4}$ in. angles. In the case of the equal angles failure occurred at loads of 10.80 and 10.60 tons and in the second instance both angles failed at 5.93 tons.

EXPERIMENTAL RESULTS AND DESIGN RECOMMENDATIONS.

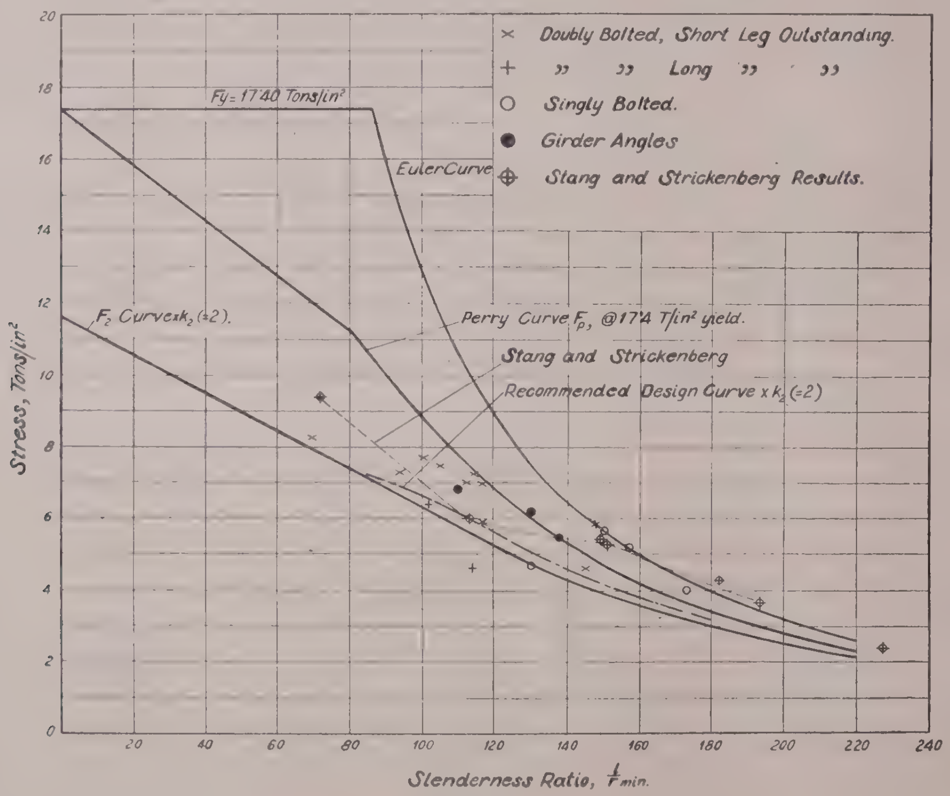

Fig. 6

General Conclusions

1. Strain distribution across the section was always linear and gave linear stress distribution up to the elastic limit. Within the elastic limit the ratio $\frac{\text{max. stress}}{\text{mean stress}}$ approximated 2.43 for fully-bolted struts and ranged from [illegible] to 3.7 for those with singly-bolted ends. In the latter case unequal angles with the short leg outstanding were found preferable to equal angles having the same cross-sectional area, whereas for heavier members requiring more than one bolt the reverse was true.

2. Failure of the struts could be explained on an elasto-plastic basis. As buckling proceeded the central moment increased but the resultant moments at the ends due to a combination of load eccentricity and joint restraint, first increased, then decreased, and finally became reversed.

3. Due to the load being applied almost on the minor axis of the angle struts, initial bending occurred about the major axis. As the loading increased the direction of bending rotated until final buckling took place about the minor axis.

4. Yield load and ultimate load were not necessarily synonymous. The ratio of these loads varied with the slenderness ratio, shape factor and eccentricity parameter, being greatest for small $\frac{l}{r}$ values and decreasing with increasing $\frac{l}{r}$ values. At values of $\frac{l}{r}$ above 160 it was found to be sensibly unity.

5. The present method for estimating failure loads of angle struts connected by two or more bolts at each end is conservative but for large slenderness ratios the Perry Formula gives a sufficient degree of accuracy. Reasonably good agreement with the experimental results is obtained when failure is computed from the formula

$$\left[\frac{fa}{F_p} \quad \frac{(f_{bc})^2}{F_y}\right] \qquad \frac{fa}{F_p} + \left(\frac{f_{bc}}{F_y}\right)^2 = 1.$$

Recommendations for Design of Single Angle Struts

A. SINGLE-BOLTED CONNECTIONS

(i) From consideration of deflections and extremes of fibre stress these are not to be recommended.

Since such members will commonly have a slenderness ratio above 150 the Perry Formula, which errs on the safe side, may be used.

(ii) The effective length for design purposes should be taken as the distance between the end connecting bolts and eccentricity of loading should be allowed for as specified in B.S.S. 449: 1948, Appendix D.

3. CONNECTIONS WITH MORE THAN ONE BOLT

(i) Angle struts with long leg outstanding are not to be recommended since the ratio max. stress/mean stress is greater than for normal fastenings and the critical load is correspondingly reduced.

(ii) Effective length should be taken as 0.8 of the actual length between centres of end fastenings.

(iii) Effective eccentricity about the major principal axis U-U may be taken as 0.8 of the actual eccentricity computed on the assumption that loading is applied at the centre of thickness of the attached leg along the gauge line of the connecting bolts.

RATIO, $\frac{max.}{yield}$ LOAD: SLENDERNESS RATIO

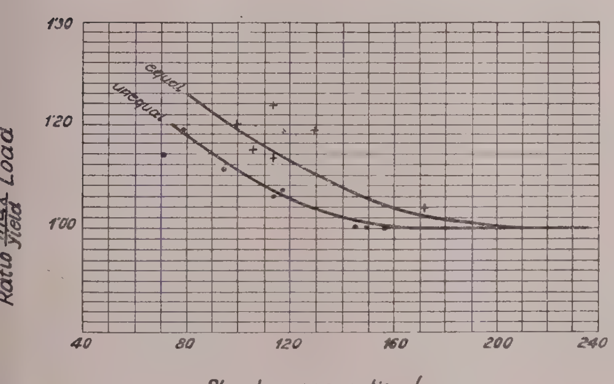

Fig. 7

(iv) Eccentricity about the minor principal axis V-V may be taken as sufficiently accommodated by the factor η in the Perry Formula.

(v) Failure may be estimated from the formula

$$f_a = \frac{F_p}{\left[1 + \frac{F_p f_a}{F_y^2}\left\{0.80\,\frac{a_1 E_B}{r^2_{max}}\right\}^2\right]}$$

here each term has the significance shown in Appendix

This gives the critical stress which first causes yield d in more general form may be written as

$$\frac{f_a}{F_p} + \left(\frac{f_{bc}}{F_y}\right)^2 = 1$$

(vi) A load factor of 2.0 against failure should be adopted. This should be applied to the value f_{cr} determined as in (v) and should vary such that

$$\text{load factor} = \frac{2.0}{\dfrac{(\text{critical stress at collapse})}{(\text{critical stress at yield})}}$$

The critical stress ratio may be taken as varying in linear fashion from 1.00 at $\frac{l}{r}$ min. = 160 to 1.20 at $\frac{l}{r}$ min. = 60.

APPENDIX A

Nomenclature

δ_L = Deflection perpendicular to the minor principal axis, called the lateral deflection.

δ_B = Deflection perpendicular to the major principal axis, called the bending deflection.

M_v = Bending moment about the minor principal axis.

M_u = Bending moment about the major principal axis.

E_L = Applied eccentricity perpendicular to the minor principal axis.

E_B = Applied eccentricity perpendicular to the major principal axis.

e_L = Observed effective eccentricity perpendicular to the minor principal axis.

e_B = Observed effective eccentricity perpendicular to the major principal axis.

r_{min} = Radius of gyration about the minor principal axis.

r_{max} = Radius of gyration about the major principal axis.

l = Effective length.

a_1 = Maximum compressive fibre distance from major principal axis.

F_y = Average tensile yield stress for specimens tested = critical bending stress = 17.40 tons/in.^{2}.

E = Average Young's Modulus for specimens tested.

F_e = Critical Euler buckling stress.

F_p = Critical buckling stress from Perry formula.

F_2 = Critical buckling stress for eccentric loading based on Table 8 B.S.S. 449 (1948).

fcr_1 = Average stress in section at yield load.

fcr_2 = Average stress in section at buckling load.

f_{bc} = Compressive bending stress due to eccentricity of loading about the major principal axis.

fa = Computed stress in section at load causing yield on basis of recommended formula.

η = $.003 \dfrac{1}{r_{min.}}$

NOTE : δ_L, δ_B and e_L, e_B are considered positive when they give rise to positive bending moments Mu, Mv. For bending moment convention, see Fig. 3.

References

[1]Stang and Strickenberg. Results of some Compression Tests on Structural Steel Angles—Technical Paper No. 218, Bureau of Standards, Department of Commerce, U.S.A., 1924.

[2]Mackey and Williamson. Experimental Investigation of a 33-ft. span Lattice Girder—International Association of Bridge and Structural Engineers, Zurich, 1951.

[3]Mackey and Williamson. Report on Experimental Investigation of Two Mild Steel Lattice Girders—B.C.S.A. Publication No. 7, 1953.

[4]Bleich, F. Buckling Strength of Metal Structures—McGraw-Hill Book Company, 1952.

Crane Gantry Girders for Steelworks*

Discussion on Paper by Mr. J. S. Terrington and Dr. J. M. Hawkes

The CHAIRMAN (Lt.-Colonel R. F. Galbraith, M.C., B.Sc.(Eng.), M.I.Struct.E., A.M.I.C.E.), welcomed on behalf of the Institution of Structural Engineers the members of the British Iron and Steel Research Association and of the Engineers' Group of the Iron and Steel Institute who were present, and introduced the authors.

Mr. Terrington then presented the paper, after which the Chairman proposed a hearty vote of thanks to the authors and declared the meeting open for discussion.

Mr. JOHN MASON (Member of Council), added his congratulations to Mr. Terrington and Dr. Hawkes, and expressed his admiration of their paper.

What he welcomed so much about the paper was that it continued and enlarged on something which was very dear to his heart and which he believed was first proposed by way of Code 113, "The Structural Use of Steel in Buildings." namely, the linking of the flange and web stresses to the elastic stability equation. When the B.S. 449, which he still thought of as Code 113, for it was much the same thing, was put forward, those responsible for it had tried to achieve this, but the difficulty was to introduce new ideas in Code form and to keep them sufficiently simple to be of use to designers. The authors of the present paper had carried this a stage farther. Mr. Terrington made the point that more careful calculation would permit even greater stresses.

Regarding flange stresses, Mr. Mason drew attention to Fig. 4 in the paper, and said that the thing about it which pleased him so very much was not only that by further calculation higher stresses could be permitted, but also, for a first approximation, the horizontal lines of B.S. 449 were a pretty good first guess for the average of the stresses which were permissible.

Web stresses were very complicated, as could be seen from the diagrams in the paper. But he asked the authors if it were not a fact that, even where a girder had reached the stage at which the web had commenced to wrinkle, there was still a large load-carrying capacity remaining before complete failure took place. So that even if B.S. 449 appeared on theoretical grounds to be a little on the high side, it was because allowance had been made for this extra carrying capacity.

Mr. Mason concluded by commending the paper to the members of the Institution and the other bodies represented at the meeting, and by commenting on Mr. Terrington's remark that he hoped more precise theoretical methods would be substituted for what had been previously rough-and-ready approximations. Mr. Mason agreed with the desirability of that, but the difficulty he saw was that if a designer were to keep his sense of proportion, he must avoid getting bogged down in a mass of figures. Mr. Mason was against the use of design by graphs, unless the person using them knew how they were derived. That was a difficulty which all those who had made Codes had felt, and he was sure that Mr. Terrington felt it. He sympathised with the authors in that difficulty, and said that if they could do anything to bring their formulæ and graphs within the day to day cognisance of the designer, they would be doing a great service to structural engineering.

Mr. O. A. KERENSKY (Member), first joined with Mr. Mason in welcoming the point made about the weakness in the present method of design. He did not agree, however, that the authors had gone far enough, for there were already experiments or theories which had overtaken them. He went through them point by point and made suggestions.

Starting with the theory and dealing with flanges, he said the authors were right in stating that B.S. 449 was only approximately correct and was applicable only to beams of medium slenderness (say R.S.J. 16 in. × 6 in.); it was never intended to apply to deep plate girders. The formula was conservative for stocky beams and optimistic for the slender ones.

Dr. Flint, of Bristol, Dr. G. Winter, of Cornell University, and others had developed and published during the last few years formulæ which dealt with beams of all slendernesses having equal, unequal and variable flanges. Dr. Flint, Mr. Allen and himself had combined these into a simple expression of the form :

Allowable stress in flange of largest lateral inertia =

$$\frac{100{,}000}{\left(\frac{l}{r}\right)^2}\left\{\sqrt{1+\frac{1}{20}\left(\frac{l}{r}\right)^2\left(\frac{t}{d}\right)^2}+K_2\right\}$$

where d = overall depth of girder
t = effective thickness of flange

involving $\dfrac{l}{r_y}$ and $\dfrac{d}{t}$ only, and covering every variety of beam.

A table could be produced which gave allowable stresses for girders with parallel equal flanges of constant section of different slenderness and different rigidity.

Allowances for flange curtailment are made by suitably reducing the t in the $\dfrac{d}{t}$ ratio, and there was an additional coefficient (K_1) which gave the t for the variations of flange. Allowance for unequal flanges were made by algebraically adding to the allowable stress for equal flanges the value of :

$$K_2 \times \frac{100{,}000}{\left(\dfrac{l}{r_y}\right)^2}$$

where K_2 is a coefficient depending on the amount of inequality and is positive when the compression flange is larger and negative when it is smaller than the tension flange.

Up to $\dfrac{l}{r}$ of about 100 (i.e., say 24 × 7½ simple beam

*Read before a Joint Meeting of the Institution of Structural Engineers, the British Iron and Steel Research Association and the Engineers' Group of the Iron and Steel Institute at 11, Upper Belgrave Street, London, S.W.1, on Thursday, October 22nd, 1953. The President (Lt.-Colonel R. F. Galbraith, M.C., B.Sc.(Eng.), M.I.Struct.E., A.M.I.C.E., in the Chair. Published in THE STRUCTURAL ENGINEER, Vol. XXXI, No. 10, pp. 268-285.

of 12 ft. 6 in. span) there was no danger of the flanges buckling and the allowable stress should equal :

$$\frac{\text{Yield stress}}{\text{factor of safety}}$$

i.e., say 9.5 tons per sq, in. for mild steel.

That compared very favourably with the old formula and would result in a great saving of weight.

Coming to the web, he said he was very much disturbed by the authors' treatment of it. By using a factor of safety of 1.5 against buckling they had arrived at a maximum permissible ratio of $\frac{\text{depth}}{\text{thickness}}$ of the stiffened web of 130. They themselves had quoted various authorities permitting $\frac{d}{t}$ ratio of 170 and more ; those girders had stood the test of time and he had never heard of one buckling. A $\frac{d}{t}$ ratio of 70, for unstiffened webs had been tested again and again and the webs never buckled, and yet the authors recommend a limit of 30. He suggested that to go back to such low limits would be a retrograde step, and that on the contrary, the ratio for stiffened webs should be extended to 240 or even more (as accepted by the B.S. Committees). In many cases the buckling of the web was not a criterion of failure, and was of no consequence until it became unsightly. In practice, every web plate which left the fabrication shop was buckled before ever it was loaded. Load factor against collapse, as in struts, should be used, instead of a safety factor against buckling. A formula on that basis had been developed by Dr. Brown and the B.S. Committee.

Further, the authors had suggested that the principal stress in the web should not exceed the maximum working stress for the material. That would mean that when using a bending stress of 9 tons per sq. in., there was only about 2 tons per sq. in. left for shear, or when using a shear stress of 6 tons per sq. in. there was only about 5 tons per sq. in. left for bending. There must be thousands of girders in use having maximum shear and maximum bending at the same section, designed at near the allowable stresses, and again he had never heard of a failure due to that. It must be pointed out that the principal stress was not the criterion of failure. The accepted theory of failure to-day was that of Mises-Hencky. For bending stress of 9.5 tons, and shear stress of 6 tons per sq. in. the equivalent stress by that theory would equal about 14 tons per sq. in., and he submitted that that was permissible when everything was taken into consideration. Stresses of approximately such magnitude were reached in truss members very often as allowable stresses are increased by 25 per cent. for wind, and by another 20 per cent. for secondary stresses, which can in fact be greater than this, resulting in fibre stresses of 14 tons per sq. in. in tension members.

The authors also gave values for inertia of stiffeners necessary to prevent web buckling. That was the correct approach to the design of intermediate stiffeners, but the values given were those developed by Timoshenko, and Mr. Kerensky was afraid that Timoshenko's values in practice were too small, particularly for wide-apart stiffeners. Moore and others in their war-time researches in America had given experimental values to the minimum inertia, and the new B.S. 153 would recommend :

$$I = \left(1.5\ \frac{d^2}{s^2}\right) \times dt^3$$

Coming to the details of design, he said he could not agree more that a single web design was the best for practically every type of plate girder, but in the case of gantry girders the unequal flanges were economic and were the proper flanges to use. The simple analysis of the authors' was a little tricky when one was concerned with unequal flanges. They had dealt with this major problem only in a footnote (on page 270), and he was not quite sure that this was adequate.

The formula for the economic depth of girder was very welcome, but presumably it applied only to girders of mild steel of slenderness ratio not exceeding 100. Actually the least weight was obtained when the total weight of flanges approximately equalled that of the web plate, and therefore the economic depth depended not only on the load per ft. and span, but also on the working stress. Economic girder depths in practice varied from about 1/6th to 1/25th, and he suggested that the values given by the formula would be on the shallow side, which for crane girders was not desirable. For economy he would use considerably deeper girders than the authors had advised, which would also be much more rigid. In crane girders that was of paramount importance. Perhaps a higher coefficient could be used in the formula.

In Fig. 8 (page 273) the authors gave a graph showing a safe ratio of $\frac{d}{t}$ for different stresses, on which stresses went up to 220 tons per sq. in., and he was not quite sure about its significance and what safety factor had been used in producing it. He believed the usual recommendation of 16 t., was still quite a good one.

Typical figures were shown on page 277, a very ingenious variety, but Mr. Kerensky suggested that some were bad. He referred to Nos. iii, v and vii as very wasteful flanges, although perhaps they were used occasionally. Again, he asked the authors whether their formulæ did apply strictly to the various fancy flanges.

With regard to joints, there were theoretically advisable positions for them, but hardly ever were they controlled by this, but usually by physical considerations, and satisfactory splices could be developed for any section of the girder.

He welcomed the authors' suggestion concerning the supporting of beams at the ends. Provision for rotation at the bearings was very desirable indeed in all girders designed as simple beams, but, as the authors had rightly pointed out, that was a problem more often met with in bridge design than in gantry girders, which usually sat on columns, which were sufficiently elastic to permit small angular changes at the ends.

What did the authors do about lateral sway ? That was one of the major problems in crane gantry girder design, and he suggested that they did not give it sufficient attention in the paper. Unless the whole flange were fully braced externally it was subject to considerable lateral bending and a special treatment was required to arrive at the safe stress. Did the authors propose to deal with that by ensuring that

$$\frac{f_b}{F_b} + \frac{f_a}{F_a} = 1$$

or could they say what modifications would be necessary to their graphs and tables ? Also, as mentioned by the authors, stiffeners have to be attached to the rail bearing flange to prevent twisting in the vertical plane and these stiffeners have got to be designed to resist this twist and also the lateral sway forces on the flange. Inertia values given in the paper would be insufficient for this purpose.

The authors' idea of efficiency in design was that yield and buckling should be reached together and that, therefore, girders were most efficient when designed at 9.5 tons per sq. in. in bending. That could not always be true, and slender girders working at as little as 3 or 4 tons per sq. in. might be lighter when set to do a definite task. If the authors' idea of efficiency were correct, no slender strut could ever be used efficiently ; yet there were seldom any other struts used in practice.

Finally, Mr. Kerensky gave a word of warning with regard to fatigue, which the authors had not mentioned. There had been some failures of crane gantry girders due to fatigue. In the case of girders continuous over several supports, i.e., subject to reversals of stress, the allowable stresses should be reduced if there is any likelihood of the maximum load being applied half-a-million times or more and even in simple girders, the possibility of fatigue effects cannot be excluded.

Mr. W. C. Brown, commenting on the proposed design methods for webs, said that, when designing the mild steel web of an ordinary rolled steel joist or a plate girder with a depth to thickness ratio of less than 85 (i.e., below the buckling range), it is usual to adopt a working shear stress of 6 or 6.5 tons per square inch, giving a factor of safety against yield of about 1.5. When webs of greater depth to thickness ratio are used the possibility of buckling arises, and in this case the authors proposed to base the safety factor not on yield, but on the theoretical critical load for a simply supported plate.

Theoretical and experimental investigation shows that

$$\text{if the value of } \left(\frac{\text{Load which causes permanent buckling}}{\text{critical load}}\right)$$

be plotted against depth to thickness ratio, a relationship as shown in Fig. 1 is obtained. Thus a web d/t =

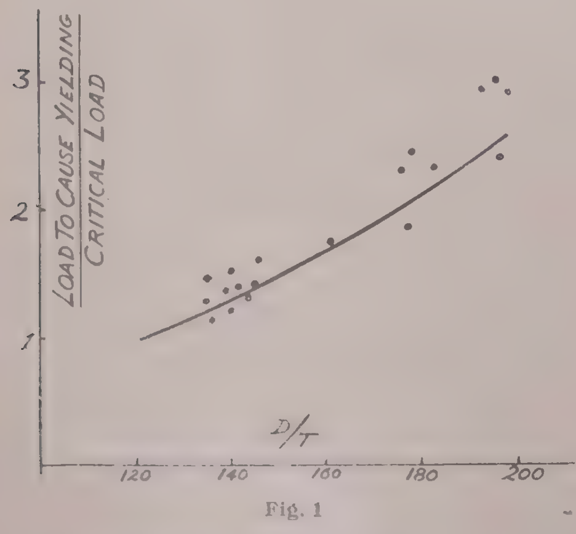

Fig. 1

180 could safely carry more than twice the critical load before producing local yielding in a small area of the web plate. He therefore suggested that the only reason for designing on a critical load criterion would exist if the lateral deflection of the web was considerable, and if one could in fact see buckles forming in the web. The lateral deflections of the web for loads in excess of the critical value are shown in Fig. 2 where again experimental and theoretical results are compared. It is seen that in practice the web deflects laterally as soon as the web is loaded, the amount depending on the initial irregularities of the plate. Near the critical load value the slope of the curve tends to increase, but no definite stability limit is usually observed, and for greater loads theoretical and experimental results are in closer agreement, with most of the experimental results lying below the theoretical curve. The lateral deflection at the centre of a web would be approximately equal to the thickness of the plate for loads twice critical, i.e., for

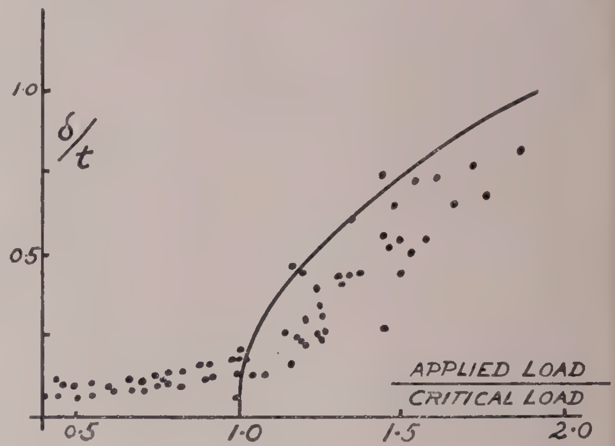

Fig. 2

a web 100 inches deep and ½ in. thick stressed to 6 tons per square inch, the deflection would be ½ in., i.e., 1 in 200.

If this deflection were accepted it became illogical to base a design on theoretical critical shear stresses and would be logical to base working stresses on the stress necessary to produce permanent buckles.

Normally, as a plate girder is loaded, the web begins to deflect and in addition to the shearing stress present, bending and membrane stresses are also set up, due to the bowing outwards of the plate.

These additional stresses are small, and it is possible to combine them with the shear stress to evaluate an "equivalent" shear stress, i.e., the combination of stress as would be produced by a single shear stress.

Fig. 3 shows such a relationship between the applied and equivalent shear stress for loads above the

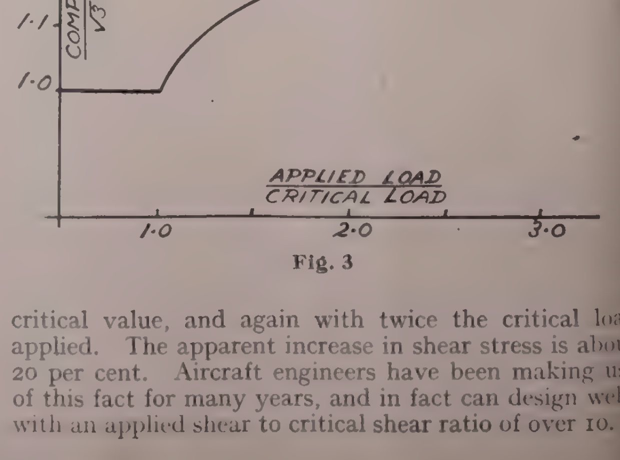

Fig. 3

critical value, and again with twice the critical load applied. The apparent increase in shear stress is about 20 per cent. Aircraft engineers have been making use of this fact for many years, and in fact can design webs with an applied shear to critical shear ratio of over 10.

Structural engineers should also make use of this relationship, and formulate allowable shear stresses in webs which are greater than those suggested by the authors, providing a constant load factor against permanent buckling as contrasted with a constant load factor against theoretical buckling.

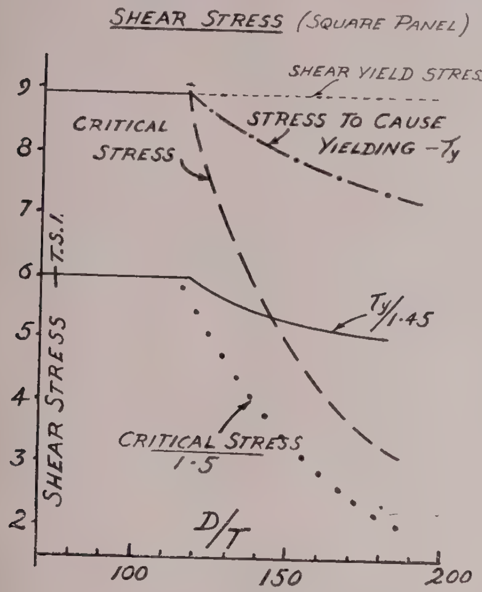

Fig. 4

Fig. 4 shows how with the use of Fig. 3 and he critical stress curve, it is possible to produce a curve or shear stresses which provides a constant load factor gainst local yield.

The middle curve is the theoretical critical stress for simply supported square plate and the shear yield stress or mild steel can be taken as just under 9 tons per

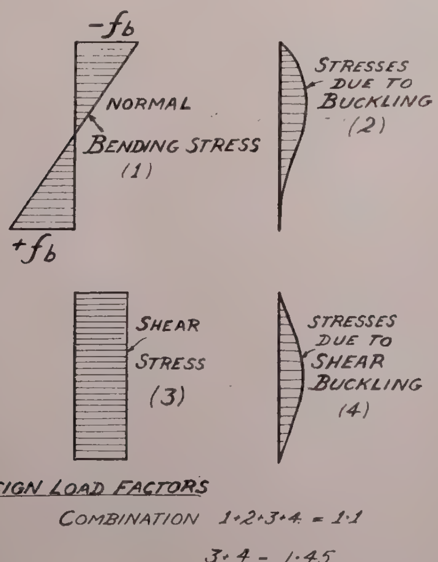

Fig. 5

quare inch. The top curve coincides with the critical ress curve at the yield stress, i.e., for d/t = 110-120, nd exceeds its value for all higher ratios. In the case d/t = 180, the critical shear stress is exceeded by pproximately twice at 7.3 tons per square inch, and sing relationship shown in Fig. 3, the equivalent crease in stress is 20 per cent. (i.e., at this load the yield stress would be reached in a small portion of the web plate near the centre). Dividing this value by 1.45 gave a design stress which maintained a constant load factor against permanent buckling. Stresses for all values of d/t are shown on the diagram.

The bottom line is the authors' suggestion, where the load factor against theoretical buckling is 1.5 and for webs of depth to thickness ratio 110 also gives a factor against permanent buckling of 1.5 and for d/t = 180 a factor of 3 which to his mind was inconsistent.

Fig. 5 shows the effect of combined bending and shear. It shows the normal bending stress in the beam and the stress distribution when there is no buckling. When bending and buckling stresses occur in very slender webs, the bending and membrane stresses due to buckling are still small and as previously described for conditions of pure shear, these stresses can be combined to produce equivalent stresses.

When normal bending stresses and the total bending and membrane stresses due to buckling together with the shear stresses are combined, a load factor of 1.1 against local yield should be maintained under the worst possible loading (i.e., the factor attaining in rolled steel joists when bending and shear stresses of 9.5 and 6.0 tons per square inch respectively are combined).

The allowable shear stresses shown in Fig. 4 are reduced particularly for the more slender webs, in such

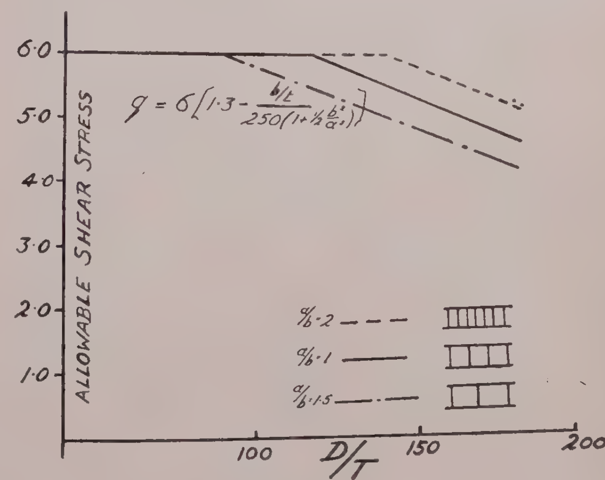

Fig. 6

a way as to make the latter combination possible and the final straight line relationship between allowable shear stress and d/t obtained as shown in Fig. 6.

These design stresses are similar to those proposed in the draft B.S. 153 and will lead to the use of more slender webs, and bring about considerable economies. Finally, on the question of stiffeners, Mr. Brown said that research work by R. L. Moore, of the American Aluminium Corporation (published in the N.A.C.A. Report No. 862, 1942) gave inertias in excess of those proposed by the authors, and mentioned that Dr. S. R. Sparkes had also noted in his paper to the British Welding Research Association that the values given by the theoretical considerations of Timoshenko were rather low.

Mr. F. Micklethwaite said that a very important factor which was ignored completely by the authors and also by Mr. Kerensky was the effect on lateral stability of the method of application of the load. In crane girders the load was applied above the top flange, not at the neutral axis as was assumed in driving the present

formulæ. As a girder tended to become unstable, deflecting laterally and twisting over, the load on the top gave a couple which increased the tendency to twist and so the girder was more unstable. The present formulæ, although often conservative, could be unsafe, particularly when applied to simply supported girders of built-up I section loaded in this way.

It had been found necessary to re-design girders designed to satisfy the regulations when they were rigorously checked for stability.

Mr. Micklethwaite did not agree with the authors on the question of using stiffeners singly and on alternate sides. There was one loading condition which was not normally taken into account, but which, it was found from experience, did frequently occur. When cranes ran skew, as they often did, the flanges of the wheels tended to ride up on the edge of the rail, so that the wheel load moved out to the edge of the rail, giving a turning moment which must go down through the rail and through the stiffeners. This introduced high stresses in the welds joining the stiffeners to the flanges, which he agreed with the authors, should be on the outside edge of the stiffeners ; this was a fatigue stress condition which could lead to failure and was much more severe with single than with double stiffeners.

Expressing agreement with Mr. Mason that it was not a good thing to publish a Code which involved a lot of graphs, and so on, which the average user did not understand, he suggested it would be very useful to issue a separate publication explaining how the Codes of Practice were actually built up.

Mr. D. ALLEN (Graduate), thought it would have been interesting if the authors had explained more fully the derivation of the many formulæ presented in the paper. It would have been of interest to know that Equation (5) gives the flange stress (with a load factor of 2) when a simply-supported symmetrical girder fails by elastic instability under a uniform bending moment. The loading of a crane girder was usually as a point load on the top flange. For a girder with a central point load applied at the centroid of the section the values given by Equation (5) may be increased by about 35 per cent. But the top flange loading decreased the stability of the girder, as mentioned by Mr. Micklethwaite, combining these two effects, it appeared that Equation (5) gave results approximately true for crane girder loading.

The value of the torsion constant J given at the bottom of page 270 was an approximation and was St. Venant's equivalent ellipse formula, which assumes that the section considered is solid, approximately symmetrical, and has no re-entrant angles. Although these conditions were not fulfilled by a normal girder, it so happened that for stocky R.S.J.s there was fair agreement, but for a deeper girder the approximation gave values considerably on the low side. For example, if used on one of the 60 in. deep girders from p. 276 it gave a value of J $\frac{1}{4}$ that of the more correct one (from Orr's formula : $J = .42\,\Sigma bt^3$).

The permissible stress was not very sensitive to the torsion constant, especially for the deeper girders, but Equation (8) incorporating the approximation, would give value 50 per cent. low (on the safe side), if applied to a 60 in. deep girder.

At the bottom of page 270 the authors gave a rule for applying Equation (5) to asymmetric girders which would give very conservative values for the case where the larger flange was in compression, but if used where the smaller flange was in compression would lead to unsafe values. Although only put forward as an illustration, a value nearer the accurate one would be obtained by multiplying Equation (5) by $\frac{0.5}{0.x}$ which showed that the stability of a girder was increased by making the compression flange large (which was a common crane girder case).

Fig. 5 in the paper showed that for a deep girder (called a "slender" section which could be confused with the slenderness ratio $\frac{L}{k_{yy}}$) K_1 could be less than 1, and so Fig. 6 should strictly show curves with K_1 less than 1.

The authors' conception of the efficiency of a section was misleading and must not be confused with economy. If a section had a permissible stress less than 9.5 tons/inch² then, supposing the flanges could not be broadened further, the only method of achieving 9.5 tons/inch² was to reduce the depth of the web. But this led to a decrease in the modulus of the section. These two effects of increase of stress and decrease of modulus when combined showed there was no saving as a result of striving to work at 9.5 tons/inch². Mr. Allen said that as an example he had taken one of the authors' girders from page 276 : web 60 in. × $\frac{1}{2}$ in., flanges 20 in. × 2 in., and worked out a span to give a permissible stress of just 9.5 tons/inch² using the draft B.S. 153 formula which was based on Equation (5) and showed good agreement. Then keeping the web the same thickness but increasing the depth to 72 in. and designing flanges as that the new girder resisted the same moment over the same span as the first example, he found the required flanges were 20 in. × $1^{13}/_{16}$ in. The permissible stress was now 8.4 tons/inch² and this section showed a slight saving over the first one.

Mr. J. MCHARDY YOUNG (Member), said there was much in the paper which gave food for thought, and it was really refreshing to see so much of it based on the principles of elastic stability.

He devoted his remarks, however, to the question of webs and web stiffeners, and he was afraid he must disagree with the authors. They began by limiting the $\frac{d}{t}$ ratio to something like 130, against the present rule of 170-180 ; and in the draft revision to B.S. 153 it was 240. He felt that to limit the ratio to 130 or 140 meant just throwing away steel.

Later in the paper the authors had referred to the use of horizontal stiffeners, though they had not developed the point. If we considered a web plate which was divided into individual panels by means of horizontal and vertical stiffeners, the buckling was very much reduced.

At the end of the paper the authors had given an extensive bibliography, but there were some surprising omissions from it. Mr. McHardy Young drew attention to the paper by Wästlund and Bergmann[1], of Stockholm, dealing with the experiments on the plates of girders, where they had made the point that in no case was the deflection and buckling of the web comparable with the thickness of the plate itself. Again, they had drawn attention to the very large reserves of strength in a web plate against buckling, which was due to a great extent to the development of membrane stresses.

Coming back to his original point, the strength of the web plate, he referred to the formula on page 277 of the paper by Mr. Terrington and Dr. Hawkes, which was obviously based on Timoshenko's theory ; they had introduced another refinement, however, in the form of a coefficient which in his opinion was quite unnecessary. Once we had determined the Eulerian stress in

a web panel, then by using a suitable coefficient, depending on the type of loading and support and the relative dimensions of the panel, we could arrive at the critical stress in either shear or bending as the case might be, which divided by the factor of safety gave the safe stresses (unless yielding is not the determining factor).

On page 278, para. 3, they gave a formula for combined shear and bending. He did not know its derivation, but it seemed to him to be wrong. Once having established the safe stresses in bending and shear we could use the theory of Huber V. Mises Hencky or Chwalla and apply the rule that the sum of the squares of the actual over the permissible stresses must not exceed 1 ft. ; this was a simple rule and it was perfectly safe to use.

On the question of stiffeners, he considered it a very good thing that they should be based on the ratio of the stiffness of the stiffeners to the stiffness of the plate. There were various schools of thought on the value of the relative stiffness. Coming back to horizontal stiffeners, however, to which there was a very brief reference in the paper, he said that if the authors had gone into the matter more fully they would have found much information in the work of Dubas[2] and Massonnet[3] and there were also the German Regulations DIN 4114[4]. The point should have been brought out in the paper that by using horizontal stiffeners it was possible to reduce the web thickness and at the same time to increase the safe stress, thereby effecting economy. He invited the authors' views on these points.

References

1 "Buckling of Webs of Deep Steel I Girders." G. Wästlund and S. G. R. Bergmann. Stockholm, 1947.

2 "A Contribution to the Study of Buckling of Stiffened Plates." C. Dubas. Third Congress Int. Assoc. Bridge and Struct. Eng., Liege, 1948.

3 "The Stability of Webs of Girders Provided with Horizontal Stiffeners and subjected to Bending." C. Massonnet. Int. Assoc. Bridge and Struct. Eng., 1941.

4 DIN 4114. Slabilitaetsfaelle (Knickung, Kippung, Beulung) Pt. 1, Sect. 17.

Mr. Terrington, replying to some of the points raised, first thanked all who had taken part in the discussion, and said Dr. Hawkes and himself were encouraged by the great interest shown in the paper.

After thanking Mr. Mason particularly for his encouraging remarks, he said the average struck by B.S. 449 was undoubtedly a good average for British Standard sections, but possibly it was not so for the more slender types.

The contribution by Mr. Kerensky was also valuable. With regard to the formula mentioned, that had come along after the paper was more or less produced, so that whilst they had become aware of it ultimately, they were not able to tie up their ideas directly with it in the time available. They did not skip the asymmetrical form with larger flanges top and bottom ; they had made the suggestion for adjustment at the bottom.

Lateral surge was, of course, a well-known effect from cranes on gantry girders, and the authors were making a practical test, in conjunction with a well-known firm in the country, with a view to re-evaluating the actual surge due to cranes and its effect on asymmetrical sections such as are employed in gantry girders.

He appreciated the point about fatigue, but he felt that surely Mr. Kerensky was pessimistic about reversals of stress. There had been variations in stresses, but he did not think actual reversals of stress.

With regard to the value of $16t$ (Fig. 8), this criterion was not a ruling factor, and the curve might have been curtailed, but it was carried on to show that the permissible stress from this consideration for projections of other proportions was quite large.

It had been said that some sections shown were wasteful, but they had some advantages where, for example, the material was disposed in the form of web coverplates to act in conjunction with the flange.

The authors were well aware of the problem of lateral surge, mentioned by Mr. Micklethwaite, although they had not dealt with it in the paper. As already mentioned, they were giving attention to it, however, and were making actual measurements on full-scale girders.

With regard to the approximation at the foot of page 270, in the value of J, he said it could be adjusted with a factor if necessary, but it was a particularly useful approximation, because it was generally applicable to all types of sections since it is only in terms of cross-sectional area and moments of inertia.

Finally, speaking of the reference made to the term "efficiency," he said that that term had been coined for the shape factor divided by the overall depth, and it seemed a reasonable description for that unit quantity because it enables sections of varying shapes to be compared directly.

Dr. Hawkes, before dealing with some of the other comments made in the discussion, said how gratified he was, at any rate at the stage so far reached, that the paper had provoked some very good discussion. From that point of view he felt it had served its purpose ; whether or not it would serve any purpose beyond that, of course, remained to be seen! After all, advances were made by discussion. One could not put down a rule as it stood and expect it to be accepted ; one needed to discuss it and to alter it as progress was made.

The problem of the buckling load on the web had been discussed a good deal of late. The authors were perfectly well aware that the recommendations they had put forward were conservative, if one considered the buckling loads on *all* girders ; i.e., a girder would continue to carry load for a considerable time after the web had buckled, and under those conditions the slenderness ratio could be quite high. But in the paper they were dealing with crane gantry girders, and the point they had had in mind right from the beginning, which perhaps was not made sufficiently clear in the paper, was that the alignment of a gantry girder was very important, and therefore they considered buckling of the web to be detrimental, even though it would not actually cause failure. So they had made suggestions, based on Timoshenko's theory. They were aware of the other theories involved, but they considered it better to be conservative in order to maintain the alignment of the crane gantry rails themselves. Then, as the formulæ became perhaps more used in practice, the allowable stresses could be increased or the slenderness could be reduced at a later date.

He believed Mr. Mason had said that the original B.S. 449 was rather played down on the theoretical side ; perhaps he was putting into Mr. Mason's words a suggestion which was not intended, but B.S. 449 had introduced a theoretical approach to what had been previously a rather practical case. The authors had felt the same way about the conditions discussed in the paper. Of the existing girders they had analysed, very few had a slenderness ratio in the web greater than 130-135 ; most of them had a slenderness ratio below 100. They had felt that if they introduced conservative figures in the paper, designers would come up to the 130-140 limit in gantry girders. When gantry girders with these

limits had been used for some years, a higher limit could be brought in.

The question of the strength of intermediate stiffeners was a complicated one. Admittedly tests had shown that the strength found from considerations of web buckling were too low. This was no doubt due in large measure to the fact that the stiffeners were supporting the compression flange as well as the web, but in all these tests failure had occurred at loads greatly in excess of the design load. Where the stresses in the web were within the limits given in the paper, the stresses in the stiffeners had been negligible. In these circumstances, therefore, the authors had felt justified in recommending strengths based on theoretical calculation, but including a large factor of safety. The fact that this factor was included in the strength given in Fig. 20 had been brought out in the original paper, but unfortunately was not made clear in the present paper.

He was glad that different methods of design had been brought out in the discussion. The formula quoted by Mr. Kerensky from the new B.S. 153 gave a similar shaped curve to Fig. 20, but with larger values. This would no doubt be the better value to use with the higher allowable web stress which Mr. Kerensky and Mr. Brown advocated.

Mr. Micklethwaite had introduced a point in the case of stiffeners placed on one side of the web only. He was not quite clear on this point, and asked if Mr. Micklethwaite had meant that stiffeners placed on one side of the web only would not be capable of resisting a moment produced by the crane wheel mounting the rail?

Mr. MICKLETHWAITE said that if there were a stiffener on one side, the couple exerted by the full load coming on to the edge of the rail would give a tensile stress across the weld on the outside of the stiffener. He had experience of a lot of stiffener welds which had cracked as the result.

Dr. HAWKES thanked him for the information and said he was not aware of it when the paper was written.

With regard to horizontal stiffeners, they were aware that the provision of them produced a considerable increase in the web strength; but they did not go further than to mention it in the paper, because they had felt that they had introduced enough food for thought already.

The authors were aware of the reference which Mr. McHardy Young had given. In their original paper they had laid stress on the saving which could be effected by the use of horizontal stiffeners, but had felt that such a subject warranted a complete paper of its own.

Finally, he said that in the time remaining he could not comment on the diagrams and tables Mr. Brown had produced, but he hoped that when he saw the written discussion he could add his own comments.

At the conclusion of the discussion the President expressed the thanks of the meeting to the authors for their replies. They were accorded with acclamation.

Written Discussion

Dr. A. R. FLINT writes: Whilst the author's suggestions for improving existing rules used in gantry design are most welcome, it might be felt that the proffered alternatives incur rather heavy design labour and the evolution of comprehensive handbooks. This is especially noticeable when dealing with the overall lateral stability of the girder. Although not stated in the paper, the basic formula presented as Equation (5) relates to a symmetrical girder under uniform bending moment and rigidly prevented from rotating about its torsional axis at the supports. Here it should be emphasised that the second term in the second bracket provides an increase in permissible stress as a result of *prevention* of warping of the section at mid-span, owing to symmetry, not as a result of warping. This restraint causes *differential* bending of the flanges when the girder is twisted.

The note concerning monosymmetrical girders is misleading. On the basis of this proposal the permissible stresses in a compression flange increase as its area is reduced relative to that of the tension flange and the above formula is still used. Owing to lack of coincidence of the centroid and shear centre of such sections, however, Equation (5) no longer applies, and must be modified if to be used in such cases. Thus, whilst on the suggested basis little may be gained by using a stiffened compression flange in preference to a stiffened tension flange, very real improvements in stability are in fact achieved with the former.

Although providing an estimate of the allowable stresses in a girder the section of which is known, the graphical determination of K_1 would prove a stumbling-block to a designer equipped with only a knowledge of span and an approximate value of depth. The authors have, however, realised the crux of the design problem, namely, to use a section which will operate at the greatest working stress of 9.5 tons/in.^{2} without reductions due to buckling. The curves presented in Figs. 11 and 12 may be used to choose suitable members, although it is difficult to see the justification for calling the shape factor per unit depth, efficiency. Surely, as all the sections are operating at the full working stress, efficiency may be only defined in relation to weight, or area, of the section.

The need to amend the existing formula given in B.S. 449 has recently been satisfied by the inclusion of a more satisfactory expression for permissible bending stress in the new Draft B.S. 153, for girder bridges. Here again the theoretical basis is given by Equation (5) of this paper, but by employing a more accurate basis for torsion constants than that due to St. Venant, and evaluating the properties of a range of sections from the smallest standard joist to girders 12 feet in depth, a very reasonable semi-empirical formula has been produced. The only parameters introduced are $L/_{K_{YY}}$ and d_1/t_1. This standard formula also contains provisions for asymmetry of sections, by means of the addition of a single term related to the ratio of lateral second moment of area of the compression flange to that of the whole section. Curtailment of flange plates is also provided for. Such modifications to the author's formula would, in contrast, produce extremely complex design curves.

On the question of loading, the authors have given no indication of the effect of application of crane loads above the shear centre of the increase in permissible stresses that may be allowed when only point loading is to be sustained. Furthermore, while figures are proposed for suitable surge loads, no indication is given of their effect on stresses. The effect of such combination of vertical and lateral loading may be considerably more serious than is suggested by simply adding the normal bending stresses computed individually. It would be desirable to provide some guidance to these crucial considerations rather than to trust that one effect will be counteracted by another.

With regard to the importance of end stiffeners and seatings it should be remembered that the proposed formula applies only when no twisting is allowed at the supports. Any freedom to rotate about the longitudinal axis necessitates a reduction in permissible stress or else an unwitting drop in safety factor. Thus the bearings

and stiffeners should prove adequate to this task as well as preventing sectional distortion.

Finally, the writer would be glad of further information on two points. The authors include a formula to decide the depth of a girder of least weight (based on distributed loading). How is it possible to decide this depth on a basis of minimum weight prior to considering the stability criteria discussed later ? Secondly, it is noted that whereas American work on web buckling would suggest a web depth to thickness ratio of 130, existing specifications permit a maximum of 170. May this discrepancy be due to the different bases used, the former being based on the stress at the onset of buckling whereas the latter may be based on the attainment of the yield stress in the post-buckling range ?

Answers to Written Discussion by Mr. Terrington and Dr. Hawkes

Replying further to some of the queries raised, the authors agree with Mr. Mason that at the commencement of web buckling there is still a large carrying capacity available, but the desirability of adequate web thickness to maintain alignment was referred to earlier by Dr. Hawkes in reply to the discussion.

Concerning points raised by Mr. Kerensky and Mr. Micklethwaite, the B.S. 153 formula relates only to a plain plate girder, and appears to give no indication of the effect of stiffening the edges and the effect of different degrees of edge-stiffening. The authors' formulæ were put forward in terms of cross-sectional area and moments of inertia precisely for this reason so that they could be applied to any type of section and not solely to an I type of section.

With regard to the possibility of stresses of 14 tons per sq. in. arising in a structure, the design load is not usually reached, and even so, such high stresses, if attained, would be purely local.

The authors appreciate Mr. Kerensky's remarks concerning webs, but wish to correct a misapprehension. In the discussion he states that "for unstiffened webs . . . the authors recommend a (slenderness) limit of 30 in." This is not so, and at the bottom of page 280 the paper actually says, "the limit of stiffener spacing is reached at a web slenderness ratio of about 90, below which no stiffeners are required for stability." This limit of 90 is somewhat higher than that mentioned by Mr. Kerensky.

Replying to Mr. Micklethwaite, the authors suggest that probably large brackets on the opposite side of the web to the plate girder would take up bending sufficiently to permit the use of single stiffeners.

The "least-weight" depth as given in Fig. 1 was, as Mr. Kerensky surmised, based on mild steel working to a maximum stress in bending of 9.5 tons/in.^{2} and with a web slenderness of 130. In these circumstances and with the assumptions made in the original paper, the depth given by the formula results in the total volume of the flanges being very nearly equal to the total volume of the web.

As Dr. Flint so clearly points out, one of the fundamental assumptions made in the development of this formula is that criteria of stability do not reduce the stresses below the maximum allowable value. To introduce the criteria of stability into such a formula is almost impossible, and as the authors pointed out in the paper, the least-weight depth is only a guide, the final depth being affected by very many other considerations.

With regard to the graph (Fig. 8) this might have been curtailed but it shows theoretically the strength of projections of various proportions.

Regarding the point of application of lateral surge referred to by Mr. Kerensky and Mr. Micklethwaite, this brings in the question of lateral surge and the treatment of torsion ; and it had been intended to make it clear at the beginning of this paper, as in the papers on which it was based, that the question of torsion was being examined and that this point was expressly excluded from the paper. It is interesting to see, however, that Mr. Allen deduces that Equation (5) is reasonably correct.

Mr. Kerensky's suggestion that beams and girders should not necessarily work to the maximum permissible fibre stress is not understood, and the authors would be glad to hear of actual examples where lower stresses are an advantage. One case is mentioned by Mr. Allen, but it is felt that his supposition that a flange cannot be widened is false.

On the question of fatigue, the authors would agree that this is in fact an argument for not raising permissible stresses.

The authors were very interested in the method of web design proposed by Dr. Brown. As was mentioned in the discussion, they are well aware that slender webs continue to carry loads in excess of that which causes buckling. Quite obviously the best method of design is one that takes account of this additional load carrying capacity. However, it was considered unwise to advocate such a design method for gantry girders. Previous practice in gantry girder construction in this country has obviously been empirical, with the result that web slenderness ratios have, in the majority of cases, been below 100. It was felt that the slow raising of this ratio would be more acceptable than the advocation of a very high limit. Furthermore, it was considered that the alignment of gantry girders was of paramount importance, and hence the factor against instability should be higher for this type of girder than for the general case. From these considerations the authors felt it was better to base the design of webs on the critical load rather than on the permanent buckling load until further practical experience had been obtained of the application of the latter design to Civil Engineering.

Coming to the method proposed in the discussion, the authors feel that Dr. Brown has made a very real contribution to the question of web design. They cannot discuss the proposals in detail as the fundamental data on which it is based are not given, particularly the origin of Fig. 3, which would appear to be the crux of the design. Fig. 3 is apparently obtained from calculations on the bending and membrane stresses set up when the plate buckles under the "applied load," but without knowing the methods used for these calculations the authors are unable to comment further. If the validity of Fig. 3 is agreed to the progression to Fig. 4 is straightforward and acceptable.

The authors are doubtful, however, of the use of so low a load factor as 1.1 for the case of combined shear and bending, and would like to know whether Dr. Brown has further justification for this figure than the one given in the discussion. The steps involved in the production of Fig. 6 are not given, but this figure is assumed to be for webs subjected to full bending stress and shear. The authors are therefore a little perturbed to find that Fig. 6 advocates an allowable shear stress for a square panel of d/t ratio 150 which is subjected to bending, of as high, if not of higher value than that given in Fig. 4 for a similar panel under shear only.

Mr. Micklethwaite's suggestion for a publication showing how the Codes of Practice were built up is agreed, and such an idea was in fact carried out by Mr. John Mason, the first speaker, in a paper read before

this Institution on the British Standard Code of Practice 113 which Mr. Mason referred to in his opening remarks.

The St. Venant approximation referred to by Mr. Allen may need an additional factor, but as mentioned earlier, its form is an advantage.

The authors thank Dr. Flint for his observations and descriptions of the B.S. 153 new draft formula. The authors feel that this formula limits the choice to a plain I type of section, whereas the ideas put forward in their paper provided for a section of any type using only the cross-section and moments of inertia. Dr. Flint's agreement on the use of the limit of 9.5 tons per sq. inch appears to refute Mr. Kerensky's claim that a girder working at a lower outer fibre stress can be efficient. The question of the effect of lateral surge and torsion raised by Dr. Flint is being further examined, and this point was made in the original papers on which this paper is based.

With regard to bearings, any means of rotation would ensure that the web is vertical at the supports, and any movement would only be in a vertical plane as permitted by a "knife-edge" bearing, and as used for a simply supported beam.

In reply to Dr. Flint's second query the difference between the two limits is purely a matter of the different factors of safety used.

Some Structural Uses of Aluminium Alloy with Special Reference to Domes*

Discussion on Paper by Mr. W. Hamilton and Mr. G. P. Manning

The CHAIRMAN (Lt.-Colonel R. F. Galbraith, M.C., B.Sc.(Eng.), M.I.Struct.E., A.M.I.C.E.), introduced the authors. Mr. Manning then presented the paper and Mr. Hamilton exhibited a number of illustrations of aluminium structures and joints and tests on aluminium.

Discussion

The PRESIDENT said he was extremely interested to see the original calculation sheet for the dome, but he presumed there were further calculations before acceptance by the District Surveyor. Again, he was interested in Mr. Hamilton's remark that the high cost of aluminium might be reduced in the near future, for he did consider that that was an essential step if aluminium were to become a really popular structural medium; unfortunately, cost would usually be the deciding factor.

Another matter of interest was to hear that the early difficulties of welding had now been overcome.

The President proposed a very hearty vote of thanks to Mr. Hamilton and Mr. Manning, which was accorded with acclamation.

Mr. L. E. WARD (Member) first congratulated the authors on an excellent design; it was possibly the neatest design in aluminium he had yet seen. He also thanked them for having illustrated the other interesting work they had done.

Coming to the paper as published, he said he was surprised at the relatively low modulus of elasticity of 9.5×10^6 lbs./sq. in. His suppliers made constant tests on the material and found that it lay between 9.8 and 10.5×10^6 lbs./sq. in. the variations being largely due to the actual composition of the alloys within the limits of the specification.

At S.M.D., he continued, they had been producing aluminium structures since 1946; they had standardised the proportions of sections, thus reducing the tedious calculations of properties. Angle sections ranged from 2 in. to 7 in. length of leg, and they had sometimes found it difficult to obtain such a comprehensive range of British Standard Sections.

The authors had said it was surprising that riveting had not been used more widely, employing aluminium rivets. In fact, they had used them for six or seven years, starting with 3/16 in. hollow and at present they were driving $\frac{3}{4}$ in. solid rivets.

Following the author's reference to corrosion resistance of aluminium, Mr. Ward illustrated the roof of a tannery (Fig. 1) which was constructed some four years ago and consisted of 55 ft. span box girders, the bottom of each girder was formed with a top hat extrusion which supported asbestos decking spanning between. By that method of construction only a small amount of aluminium was exposed to the severe atmosphere inside the building and there was no evidence of corrosion.

One of the advantages, that of light weight, was demonstrated in the Comet Flight Shed for the De Havilland Aircraft Company at Hatfield. This building, of 200 ft. span and 330 ft. long, was completed in approximately a year, and the order was secured in face of competition with steel and reinforced concrete. Mr. Ward commented jocularly that his company must possess good designers or good salesmen, for they could get such buildings erected. Fig. 3 illustrated one of the legs of the portal frame showing the riveting in progress. The bulb angles used for the structural members ranged from $2\frac{1}{2}$ in. to 7 in. Some difficulty was experienced in getting long lengths of plate for strengthening the booms, but this was achieved by using a 9 in. wide flat extrusion.

Mr. Ward was most intrigued by the tenon joint the authors used on domes. In that connection, he said that one of the criteria when designing was to cater for uplift due to wind loading. A modest wind load on such a structure, assuming also that it was not uniformly distributed, could give rise to high bending moments. If that be so, had the authors carried out any bending tests on the joint, for it appeared to be considerably weakened by cutting away half the section?

Finally, he said the sheeting appeared to be extremely thin, and he expressed admiration for the authors in regard to their use of 16 SWG sheet; especially so when 13 SWG sheeting on closer purlin centres was used on the Dome of Discovery.

*Read before a Meeting of the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1, on Thursday, December 17th, 1953, the President (Lt.-Colonel R. F. Galbraith, M.C., B.Sc.(Eng.), M.I.Struct.E., A.M.I.C.E., in the Chair. Published in THE STRUCTURAL ENGINEER, Vol. XXXI, No. 12, pp. 337-350.

Mr. R. A. SEFTON JENKINS (Associate-Member) said that he noticed that the authors had analysed the domes as thin laminar structures and neglected any bending effects that he thought were likely to arise when the domes were unevenly loaded. The analysis of a structure of this sort by relaxation or similar methods was an extremely tedious process at the best.

The authors mentioned in their paper that they considered the analysis of this type of structure by models to be impractical. Mr. Jenkins said that he had evolved a method of model analysis which was extremely useful when faced with three dimensional problems.

The problem that he had had was a series of beams crossing each other, and fixed to each other at the crossing points. This he analysed by use of the model shown in Fig. 4.

The procedure in this case was to load the model at each crossing and take a series of readings on the Moment Indicator shown in the photograph. His Practice had evolved this instrument and had found that it gave very good results, giving the bending moment at the point to which it was fixed by merely multiplying the dial reading by a constant. They had also evolved a similar instrument which gave the shear at the point to which it was fixed.

This general method Mr. Jenkins thought could be applied to the problem of the analysis of the domed structures under discussion. He suggested the following method :—

The unknown quantities were the bending moment, vertical shear and thrust in each member, under any loading. The first stage would be to construct a model similar to that shown in Fig. 4 but to the shape of the dome. By preventing any horizontal movement of the ends of the members, readings on Moment and Shear Indicators could be taken for a load placed at each crossing point. Some of these readings will be positive, whilst others will be negative, by neglecting the negative readings the worst case of loading is automatically given.

The next stage would be to analyse the horizontal thrust in the members. This could be done in the same way as when the horizontal thrust of an arch is found using a Xylonite model. This would be done in this case by moving each end of each member radially and measuring the deflection that this causes to the model. This then gives the influence line of thrust at the ends of the members.

As a refinement it might be that one wanted to know what happened when the ring girder expanded under load. This could be done by taking a series of Moment and Shear Indicator readings before and after moving the ends of the members by a known amount. The movement of the ring girder for a given thrust could either be calculated or be found by means of a separate model.

Fig. 1—Tannery *By courtesy of S.M.D. Engineers*

The results obtained previously could then be amended by these results.

In putting forward this method, Mr. Jenkins had assumed that no account had been taken of the sheeting. In practice the domes stood up to the tests required of them, and so he assumed that the sheeting played a considerable part in reducing the moments in members. He felt that the authors should be congratulated on devising such a structure.

Mr. JOHN DOSSOR (Delegate Member of Council), added his appreciation of the author's extremely interesting paper.

To confirm their remarks about the difficulty of convincing one's clients that aluminium would resist corrosion for a long time, he said that recently his organisation had submitted a design in aluminium for a railway station roof, the weight of which was a fraction of that of reinforced concrete arched slabs. Although there was interest in it for some time, the decision, on balance, was against aluminium because of the possibility of corrosion. He had seen there the great advantage of being able to get a large number of different

sections to suit just what was required, as against steel, because one could get aluminium sections extruded, although the quantity required might be small.

His organisation had also been using aluminium quite a lot in sewage disposal works. It was used for gas collectors, to replace wood in scum boards, and so on. Some had been in use for ten years and had stood up very well ; there was no corrosion in the scum boards.

Mr. M. Bridgewater expressed his great interest in the paper, particularly as, in his capacity as a Development Engineer with Northern Aluminium Co., Ltd., he had been associated with much of the work described in the paper.

Mr. Manning and others had mentioned the weathering of aluminium alloys and he thought that some recent work carried out in America would be of interest. Some

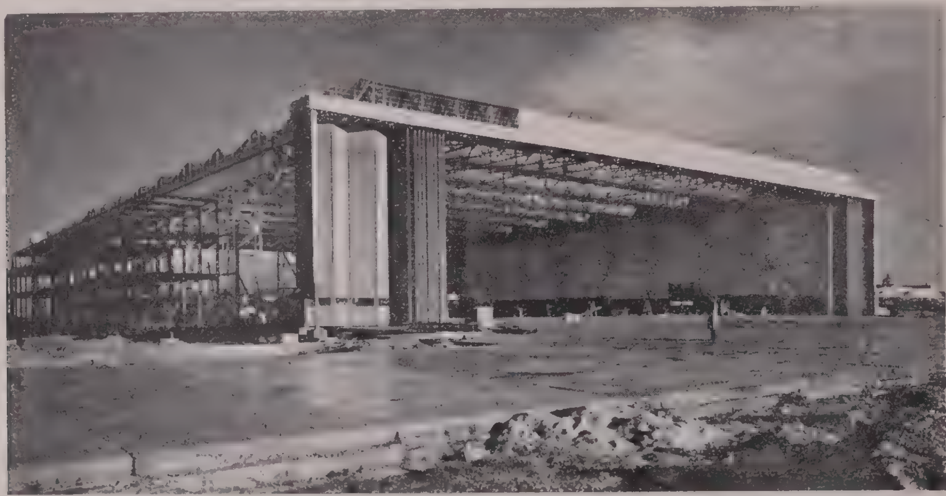

Fig. 2—Comet Flight Shed *By courtesy of S.M.D. Engineers*

Fig. 3—Portal Leg (Comet Flight Shed) *By courtesy of S.M.D. Engineers*

On the other hand, concrete got into trouble by frost, wood was troubled by distortion, and so on. Aluminium was a very useful material in sewage purification works.

One of the troubles with thin aluminium sheet was to make it strong enough to enable people to walk about on it where necessary, and some of the combinations of wood and aluminium bonded together were very useful. One could make a very light arch with some of the patented combinations of laminated wood and corrugated aluminium.

25,000 specimens had been exposed for 20 years in all types of atmosphere—industrial, tropical, marine and rural. A complete survey of these specimens had now been made and the results were very encouraging. In all cases the majority of corrosion had occurred during the first two years, after which time it proceeded at a constant but greatly reduced rate : for instance, in severe industrial environments, the pitting rate was only .08 mils/year. Using this experimental data, the life of an aluminium sheet in a known atmosphere could

be estimated and it had been shown to be vastly superior o galvanised steel and as good as either copper or zinc. For a given life of sheet, aluminium was undoubtedly the cheapest of all the durable roofing materials.

He agreed with an earlier speaker—that there was a need for some efficient aluminium sections to supplement the British Standard sections, which were not particularly efficient. The Aluminium Development Association had now produced a range of bulbed angles and lipped channels, which had been adopted as an Industry Standard and which produced substantial economies over the existing standard sections.

In the paper, there was a reference to the splitting of the hollow extruded rib section. When the first dome was constructed, the method of making hollow sections was in its infancy and very little was known about the fusion of the plastic metal within the die. A great deal of work had now been done on this method of manufacture and there was now no danger of this trouble recurring. The aircraft industry were now building the highly stressed helicopter rotor blades from hollow

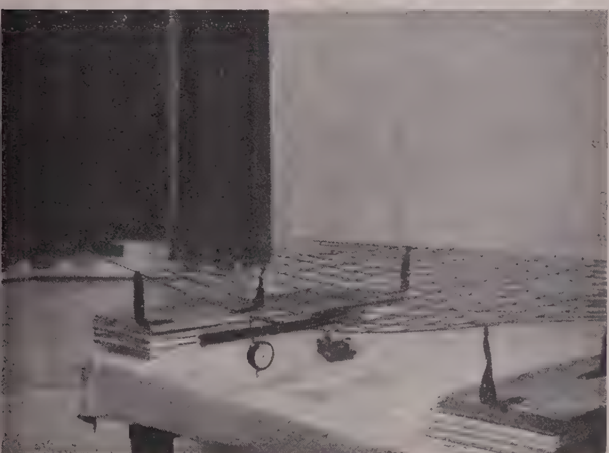

Fig. 4

extrusions manufactured by the same process, a convincing proof of the reliability of this type of section.

Finally, he said that the aluminium industry appreciated how difficult it was to pioneer a new material, and Mr. Hamilton and Mr. Manning were to be congratulated on the way they had succeeded in carrying out a major structural development. He felt that the whole of the aluminium industry owed them a vote of thanks.

Mr. T. W. Gibson (South-Eastern Gas Board), after thanking the Institution for having invited him to the meeting, said the gas industry was very interested in the suitable application of aluminium in view of the very high cost of maintenance of large steel structures.

He was rather surprised that the authors had not mentioned the use of aluminium for the moving parts associated with mechanical plant. If, for example, aluminium was applied to the mechanical shovel, horsepower could be saved and a considerable increase in the pay load would be obtained.

He also stated that his Board had already some experience of aluminium structures and they appeared quite satisfactory and economically sound in the conditions prevailing on a gas-works.

Mr. Leslie Thompson (Associate-Member) referred to an aluminium-framed structure which was fixed over a hydraulic water tank, where the conditions were very humid and steamy. It had been in use for about five years ; there was a little scale on the members after about six months, but he had not found any further deterioration since.

He asked if the authors could indicate any precautions which should be taken in fixing aluminium roof sheets to existing steel structures such as purifier sheds and retort houses.

Mr. R. J. Wilkins (Member) asked what was the meaning of the letters " L.U.D." at the top of page 339 of THE STRUCTURAL ENGINEER ?

Mr. Manning, replying to the discussion, first congratulated Mr. Ward on being able to sell aluminium hangars to anybody ; he himself had found the task quite impossible so far. Mr. Ward had made quite a lot of interesting points, not all of which called for comment.

Both Mr. Ward and Mr. Jenkins had raised a point as to what were the stresses in the domes under some kind of wind loading or partial loading ; apparently there was something he had not made clear. He drew attention to two figures. On page 344 there was a calculation estimating the maximum stress at 9,260 lb. per sq. in. in any member; on page 347 the figure given for the maximum stress as registered by the extensometers was 9,333 lb. per sq. in. If he had had any sense he would just have left those figures for readers to admire the accurate forecast, but the conditions of loading were very different ; it was only an accident ! But he drew attention to the statement on page 347 that : " Later, this dome was subjected to various arrangements of partial loading."

First the load was applied in rings, so that the outer half of the dome was loaded and the inner half was unloaded. Then there were arrangements for quarter, half and three-quarter loadings to about 30 lb. per sq. ft. It was quite clear that the total wind loading on a dome of that kind was very, very small normally. He knew there were Codes of Practice which stated definite figures, but he was quite certain that these could not apply. However, the nature of the stresses, with a certain amount of wind suction over the structure, would give the same type of stressing that was obtained with half loading. In one case there might be tension where in another case there was compression, but the stressing was of the same type, and he was quite satisfied that the stresses must be small. The possible exception was the dome at Eastbourne. It was over a reservoir situated in a hollow scooped out under the brow of Beachy Head, and the wind blowing over it might form free vortices and there would be a much heavier uplift than in any other type of situation. But the tests made had convinced Mr. Hamilton and himself that there was no trouble due to wind loads on those structures ; it was not one of their problems.

In reply to Mr. Jenkins with regard to model analysis, he said he hoped the remarks in the paper were clear. He did not mean that model analysis was useless in all cases. But they did make a model of the particular dome, and it was clear that in order to reproduce the conditions in the dome it was necessary to make the joints to an accuracy of something like one-hundredth of a thou., because in those three-dimensional problems the fitting of the joints was most important. The authors had felt that in that case, and that case only, they had tried it and could not get any sense out of the model. That was their experience.

Then Mr. Jenkins had raised the point about the outward movement of the ring. It was not a major point, but clearly, when we got up to 500 ft. it would be. We could put them up on flexible columns if we were

lucky enough to have a place where such columns could be used, but in most cases we could not use them. The authors' suggestion was to put the thrust ring on a bed of asphalt, bolt up the roof to an extent which allowed it to move slightly, and then bolt it down possibly when the dead load was 5 lb. per sq. ft. or something like that. The outward movement of the ring was a point the authors would certainly deal with on very large domes.

The eight or ten years of experience of the use of aluminium at sewage disposal works by Mr. Dossor was longer than that of the authors ; theirs was only very recent.

Commenting on Mr. Bridgewater's reference to the tests on specimens which had been going on in America for some time, he said one of the difficulties was the difference in the nomenclatures applying to the different alloys. He believed that even in our small country there were three more—the Navy had one, the Ministry of Works had one, and so on—and the Americans had another half-dozen. So that it was difficult to follow the results. Some of them might be put forward by people who were trying to sell aluminium, or possibly by people who were trying to dissuade others from buying it. But if he wanted to know how aluminium stood up to the climate he had only to look at his own "pre-fab." cat shelter standing in the garden.

On Mr. Gibson's point about the saving of weight in machinery, he said the authors had actually designed some crane jibs, but had not built any yet.

The aluminium roof over a warm water tank was at Wandsworth, he believed (Mr. Thompson agreed), and it included some of his own manual work.

About the jointing of aluminium sheets to steelwork, he said there was always the question of anodic action between two different metals. It seemed that in exposed positions, such as at the seaside, where there was a lot of salt spray, there could be very severe anodic action where the two metals touched ; but he imagined that indoors, in a dry atmosphere, there would probably be no anodic action. The question was whether the atmosphere was such as to be likely to set up that action ; if so, the two metals should be kept apart. The next question was to know what to use in order to keep them apart, and it was a difficult matter. If the bolts were painted, the paint might be damaged. We could put felt washers between the two metals. We could obtain aluminium sheeting hooks, and aluminium sheeting bolts, and possibly by their use we could get the contact between the steel and aluminium on the dry side of the roof.

In reply to Mr. Wilkins, he said the letters "L.U.D." meant limiting ultimate deflection. That was a method of designing compression members under all kinds of loads which he had evolved many years ago and had used exclusively since. He could design a strut of varying section (say) a crane jib, tapering one way in levation and another way in plan ; he had used the method for both permanent and temporary struts, timber struts, tubular scaffolding, reinforced concrete columns, and so on, and it gave him very good results, particularly where there were varying sections and transverse loads. He had written several articles on it.

Mr. Hamilton, who also replied, said that one of the interesting features of the dome was that the ribs were designed to take the compression and the sheeting was also made to work ; it was not there merely to keep out the weather.

If a local load were applied to *any* part of the dome, it should tend to distort ; but it could not do that, because the sheeting, which was running at right angles and was fixed to the permanent structure, prevented displacement. Therefore the ribs must remain in compression, and the tension went into the sheeting.

They had had the opportunity to build the first welded gas tank roof of any size in England, for the Regent Oil Company. With the kind permission of the Chief Engineer they had had it tested under gas pressure and had asked Professor Owen, of Liverpool University to make some stress measurements, for they wished to know how it was behaving. The roof was of 80 ft. diameter, and permission was obtained to load the roof under a gas pressure of 12 in. (6.25 lb. per sq. ft.). Unfortunately, Professor Owen was not able to make a complete stress analysis, but he had suggested taking a small section and testing it very thoroughly. Owing to the roof being under internal pressure, they had to have a ring beam to take the compression which resulted and it was overloaded by 50 per cent. So that they were prepared to find stresses of about 10-12 tons per sq. in. in it. It was welded to the side of the steel tank, a steel compression ring. With a gas pressure of 12 in. the compression stress in the outside ring was less than 4 tons per sq. in. Tests were made on the sheet at about 1 ft. from the periphery, and it was found that the 16-gauge sheet was actually carrying 1 ton per sq. in. in compression. So that all the roof members were doing work, whereas one would not normally expect them all to do so. The stresses were very much less than were expected.

He hoped that would answer the point that the tension in the members could not really exist because the sheeting took it out.

The sheeting problem was very important, because in such a roof the material must be kept down to the absolute minimum. If he had had his way he would have used 18-gauge instead of 16-gauge ; it was on the advice of his good friends the Northern Aluminium Company that the 16-gauge was used. But before that they had made up a frame 10 ft. square and had loaded it with 16 in. of sand. In addition, two people had stood on the centre. He was afraid that the rivets would tear through the thin sheeting, but there was nothing that he could detect which showed stress. The rivets were in tension ; he wanted to get the very worst condition, and still he could not find anything visually which frightened him. The loads were infinitely in excess of anything one would expect.

To walk on a 16-gauge sheet gave one a sensation that the sheet was too thin ; but the material was of such strength that the load from one's heel would certainly not puncture it.

With regard to corrugated sheeting, he said that a Swiss gentleman had made an interesting design of sheeting which did not require any bolts at all. It was a most ingenious method. At the moment the sheet was in coils 60-odd ft. long and only 2 ft. wide ; Mr Hamilton could not see why it should not be 4 ft. 6 in. or 5 ft. wide.

A very interesting application of aluminium was for very long piles. The cost of driving very long concrete piles was enormous, and he did not think anyone had handled very long ones without causing cracks ; although they could not be seen, he was sure they were there. There was no reason why an aluminium pile in salt water should not give the engineer all that he wanted, with very much less trouble to everybody.

The President, at the conclusion of the meeting expressed appreciation to Mr. Hamilton and Mr. Manning for their very interesting demonstration and for the way in which they had answered the questions raised.

Institution Notices and Proceedings

ORDINARY GENERAL MEETING

An Ordinary General Meeting of the Institution of tructural Engineers was held at 11, Upper Belgrave treet, London, S.W.1, on Thursday, May 27th, 1954, at .55 p.m. Lt.-Colonel R. F. Galbraith, M.C., B.Sc., I.I.Struct.E., A.M.I.C.E. (President), in the Chair.

The following members were elected in accordance ith the Bye-laws. Will members kindly note that the lections as tabulated below should be referred to when onsulting the Year Book for evidence of membership.

Students

ROOME, Gerald William, of Cape Town, South Africa.
IARRISON, Leonard Charles, of St. Helens, Lancs.
IOCZKO, Stanislaw Antoni Krzysztof, of London.
IEESHAM, Ronald, of Liverpool.
NADDON, Robert Angus Anderson, of Johannesburg, South Africa.

Graduates

THA, Derek, of Sheffield.
ALLANTYNE, Desmond Trevor, of London.
EANEY, Thomas Patrick, of London.
ROOKES, Edward Michael, B.Sc.(Civil) Durham, of Rainton Gate, Co. Durham.
URTON, Michael Arthur Brian, of Kingston upon Thames, Surrey.
HOWDHARY, Abdul Wahid, B.E.(Civil) Sind, of Karachi, Pakistan.
ONNER, Morris William, of Send, Surrey.
REASE, David Plaistow, B.A.(Cantab.), A.R.I.B.A., of Edinburgh.
OSHI, Jagdischandra Jagjiwandas, B.E.(Civil)Hons. Bombay, of London.
VANS, Glyn, of Chester.
ISHWICK, Adrian Leo, of London.
RIEDLER, Sali, B.Sc.(Civil Eng.), of Croydon, Surrey.
RIIS, Erik Juul, B.Sc.(Civil Eng.) Witwatersrand, A.M.I.C.E., of Johannesburg, South Africa.
ERAGHTY, Edwin Martin, B.Sc.(Civil Eng.) Cape Town, of East London, South Africa.
REEN, Ernest, of London.
UHA, Amitabha, B.Sc.(Gen.Sc.), B.E.(Civil Eng.) University of Calcutta, of London.
ANDA, John Desmond, of Manchester.
BREBOWSKI, Ryszard Tadeusz, of London.
HUNN, William Gordon, B.Sc.(Eng.) Rand, of Germiston, Transvaal, South Africa.
PEDDING, Alan Henry, of Liverpool.
TAVELEY, Peter William, B.Sc.(Eng.) Natal University, of London.
IRTUE, John Clive, B.E. Tasmania, of London.
EOMANS, Robert, of Birmingham.

Associate-Members

HENG TSING-KWAN, B.Sc.(Civil Eng.) Shanghai, of Hong Kong.
EKSHMANAN, Nilakantan, of Singapore.
IULGREW, Raymond Maurice, of Krugersdorp, Transvaal, South Africa.
AN CHIN THYE, B.Sc.(Civil Eng.) Canton, Lingman University, of Singapore.
AN GYSEN, Theodorus Johannes, of Heathfield, Cape, South Africa.

Associate

WARD, Ronald, F.R.I.B.A., F.R.San.I., F.I.Arb., of London.

Member

WALKER, John, of Dorking, Surrey.

Transfers

Students to Graduates

ALLMAN, Laurence Mills, of Johannesburg, Transvaal, South Africa.
DOYLE, Terence Charles, of London.
D'SYLVA, Eunan Declan, B.Sc. Bombay, of Bombay, India.
HILLIER, Derek John, B.Sc.(Eng.), of London.
MORRIS, Anthony, of Upton-by-Chester, Cheshire.
OKE, Abraham Adebayo, of London.
SAXENA, Kailash Chandra, B.Sc. Agra, LL.B., of Bombay, India.
TRAVERSE, John Derek, of St. Helens, Lancs.

Graduates to Associate-Members

BOWMAN, Charles William, A.M.I.C.E., of Toronto, Ontario, Canada.
DAVAR, Kersi S. B.E.(Civil) Bombay, of Bihar, India.,
DONALD, Allan, B.Sc.(Hons.) Glasgow, of Paeroa, New Zealand.
HARRISON, Richard Hayton, B.Sc.(Civil) Birmingham, of Derby.
LYNCH, William Henry, of Tripoli, The Lebanon.
MCCARTE, John Miller, of Glasgow.
MASTERS, Patrick Anthony, of Derby.
NOHR, Max, of Johannesburg, South Africa.
NOLLER, Gerald Rous Allen, of Wellington, New Zealand.
SALMON, Norman Derek, B.Sc. (Civil Eng.) Witwatersrand, of Johannesburg, South Africa.

Associate-Members to Members

BRESLIN, Thomas, M.I.C.E., of Johannesburg, South Africa.
BRUETON, George Raymond, A.M.I.C.E., of Cardiff.
REUBEN, Reuben Simon, F.R.I.B.A., of Bombay, India.
THROWER, Henry William, of Coventry, Warwicks.

Associate-Member to Retired Associate-Member

JOHNSTON, James Halcro, B.Sc.(Eng.), M.I.C.E., of Orphir, Orkney.

OBITUARY

The Council regret to announce the deaths of Captain DONALD GREWAR, M.B.E., R.E. (Member) and Lt.-Colonel FREDERICK JAMES EASTERBROOK, M.C. (Retired Member).

ANNUAL GENERAL MEETING

The Annual General Meeting of the Institution of Structural Engineers was held at 11, Upper Belgrave Street, London, S.W.1, on Thursday, May 27th, 1954, at 6 p.m., Lt.-Col. R. F. Galbraith, M.C., B.Sc., M.I.Struct.E., A.M.I.C.E., in the Chair.

The Secretary (Major R. F. Maitland, O.B.E.), read the notice convening the meeting.

The minutes of the Annual General Meeting held on May 21st, 1953, as published in THE STRUCTURAL

ENGINEER, July, 1953, were taken as read and were confirmed and signed.

Mr. E. Granter (Past President), moved the adoption of the Sessional Report of the Council and the accounts for the financial year, 1953. Mr. Walter C. Andrews (Past President) seconded the motion, which was carried unanimously.

Mr. L. Scott White (Past President), proposed the re-election of Messrs. James Meston & Co., Chartered Accountants, as Auditors for the ensuing year. Mr. L. E. Kent (Vice-President) seconded the motion, which was carried unanimously.

The Secretary then read the report of the Scrutineers on the ballot for the election of President, the Honorary Officers, and the ordinary members of Council for the Session 1954-55, as follows :—

To : The Council of the Institution of Structural Engineers.

Gentlemen :

We, the undersigned, report that at the request of the President we have duly carried out the duties of Scrutineers of the Ballot for the election of Honorary Officers and Council for the Session 1954-55, and we report accordingly as follows :

We received 818 Ballot Papers, of which we rejected 48 as wholly spoiled and 21 as partly spoiled. We have attached a separate sheet showing the number of votes received by each candidate.

We declare the result of the Ballot to be as follows :—

Elected

President : Dr. S. B. Hamilton, M.Sc., Ph.D., B.Sc. (Eng.), A.R.C.S., M.I.C.E.

Vice-Presidents : Mr. S. Vaughan, B.Sc., M.I.C.E., A.C.C.I ; Mr. J. Guthrie Brown, M.I.C.E. ; Professor A. G. Pugsley, O.B.E., D.Sc.(Eng.), F.R.S., M.I.C.E., F.R.Ae.S. ; Mr. G. S. McDonald, M.I.C.E. ; Mr. L. E. Kent, B.Sc.(Eng.), M.I.C.E. ; Mr. W. H. Woodcock, F.C.S.

Honorary Treasurer : Professor A. L. L. Baker, D.Sc.(Eng.), B.(Tech.), Hon. A.C.G.I., M.I.C.E.

Honorary Secretary : Mr. R. W. Schofield.

Honorary Librarian : Mr. H. C. Husband, B.Eng., M.I.C.E., M.I.Mech.E.

Honorary Editor : Mr. F. R. Bullen, B.Sc.(Eng.), M.I.C.E.

Honorary Curator : Lt.-Colonel G. W. Kirkland, M.B.E.

The above are all elected for ONE YEAR.

ELECTED AS ORDINARY MEMBERS OF COUNCIL (LONDON)

Dr. F. G. Thomas, Ph.D., B.Sc., M.I.C.E. ; Mr. J. Singleton-Green, M.Sc., M.I.C.E., A.M.I.Mech.E., M.Soc.C.E.(France) ; Dr. A. R. Collins, M.B.E., D.Sc., A.M.I.C.E.

The above are elected for three years.

ELECTED AS ORDINARY MEMBER OF COUNCIL (COUNTRY)

Dr. D. D. Matthews, M.A., D.Eng., M.Sc.(Eng.), A.M.I.C.E., A.M.Am.Soc.C.E.

The above is elected for three years.

ELECTED AS ASSOCIATE-MEMBER OF COUNCIL (LONDON)

Mr. K. Severn, M.C., M.A.(Cantab.), A.M.I.C.E.

The above is elected for three years.

ELECTED AS ASSOCIATE-MEMBER OF COUNCIL (COUNTRY)

Mr. G. S. Gowland.

The above is elected for three years.

We are, Gentlemen,

Yours faithfully,

(signed) H. BROMPTON
C. E. CANNONS
O. A. KERENSKY
H. WINGRAVE NEWELL
(Scrutineers).

On a motion by the President, a vote of thanks was unanimously passed to the scrutineers.

EXAMINATIONS, JULY, 1954

The Examinations of the Institution will next be held at centres in the United Kingdom and Overseas on July 13th and 14th, 1954 (Graduateship), and 15th and 16th (Associate-Membership).

ENGINEERING INSTITUTE OF CANADA

The Engineering Institute of Canada have agreed that Associate-Members and Members of the Institution of Structural Engineers be accepted as meeting the educational requirements of the Engineering Institute of Canada for member grade.

REPRESENTATION

The Council have re-appointed Mr. F. S. Snow (Past President) as the Institution's Representative on the Professional Classes Aid Council for a further period of three years.

RESEARCH AWARDS

The Council has instituted a Research Prize Fund, from which awards may be made annually to the author or joint authors of papers describing original research which they have carried out. Research awards may be made for papers read at Headquarters or in the Branches and published in the Journal, or for papers published in the Journal only without being read at an open meeting.

The assessment for such awards will be made annually but awards will be made only to the contributors of such papers as reach a standard judged by the Literature Committee to be satisfactory.

Work submitted under this scheme must be original and may include any of the following :—

(a) investigations of an experimental or analytical character ;
(b) studies of historical or statistical records ;
(c) improvements in principles or methods of construction ;
(d) research into methods of structural engineering and building, the nature and use of plant and the organisation of engineering work ;
(e) any related or combined studies which are deemed by the Literature Committee to be of a research character.

In cases where the research work described in the paper was not the work of one individual, the names of all the collaborators should be given in the paper.

Awards may take any or all of the following forms :— A research medal ; a diploma ; a money prize.

Application for consideration for a research award must be made to the Secretary of the Institution, and in preparing papers for reproduction in the Journal, authors must comply with the conditions laid down for all such contributions. Particulars of these conditions may be obtained from the Secretary.

In judging research papers, the following factors will be considered :—

(a) the nature of the subject and its conclusions ;
(b) the value of the paper in advancing the science and art of structural engineering ;
(c) the standard of preparation and orderly arrangement of the subject-matter.

Research papers will also be eligible for adjudication for the Institution Medals if they comply with the Regulations governing those awards.

The closing date for the receipt of applications in respect of papers published in the Journal between October, 1953, and September, 1954, is October 1st, 1954.

MACLACHLAN LECTURE COMPETITION, 1955

The closing date for the receipt of entries for the next Maclachlan Lecture Competition is Thursday, March 31st, 1955. The general conditions of the competition are as follows :

1. The Institution of Structural Engineers shall institute a written lecture to be known as the Maclachlan Lecture and to be held annually.

2. The subject of the Lecture may be on any aspect of Structural Enginering as long as in every second year the subject shall be confined to steel structures. (This will be the case in 1955.)

3. Entrance into the competition for the Lecture shall be confined to Associate Members of the Institution, who are under the age of 32 years.

4. All papers entered for the competition shall be submitted to assessors to be appointed by the Council of the Institution, and all such papers (including the prize-winning Lecture) shall be available for publication in the Journal of the Institution at the discretion of the Council.

5. No paper submitted shall have been published or read elsewhere.

6. The winner of the competition shall be required to present the Lecture to a meeting of the Institution at which he will be presented with the sum of £17 10s.

7. Should a competitor's paper be considered worthy of ranking second in merit he will receive a consolation award of £5.

8. In the event of there being no winner of the competition in any one or more years, whether because no lecture submitted is considered to be of sufficient merit to warrant award, or for any other reason, the Institution shall transfer the above sums to the Research Fund of the Institution.

Particulars of the Competition for 1955

1. The Maclachlan Lecture will be given at a meeting of the Institution to be arranged towards the end of 1955.

2. The subject of the Lecture shall be confined to steel structures.

3. The work should be submitted as the script of a lecture which the author, if successful in the competition, will deliver before an audience in the course of about one hour. The development of mathematical formulæ and detailed calculations should be avoided as far as possible in the text ; if they are essential they should be embodied in appendices. Photographs, drawings, graphs, &c., which would appear as illustrations to the lecture in published form, should accompany the script. If additional illustrations would be shown as slides, a list of these should be included.

4. Six copies of each Lecture should be submitted and should be addressed to the Secretary of the Institution.

5. The closing date for the receipt of entries by the Institution is Thursday, March 31st, 1955.

ALUMINIUM DEVELOPMENT ASSOCIATION RESEARCH SCHOLARSHIP

A research scholarship in the use of light alloys in structural engineering is offered in alternate years by the Institution of Structural Engineers in collaboration with the Aluminium Development Association.

The duration of the scholarship will be two years and the value £400 per annum. The first award will be made this year to date from October 1st, 1954.

Details and application forms are obtainable from the Secretary of the Institution. Completed application forms should be sent in to reach the Secretary by July 15th, 1954.

Notices of this scholarship have already appeared in the technical press.

LONDON GRADUATES' AND STUDENTS' SECTION

The following Honorary Officers and Committee members have been elected for the Session 1954-55 :—

Chairman : Mr. J. F. S. Pryke.
Vice-Chairman : Mr. D. S. Little.
Hon. Secretary : Mr. J. A. Pope.
Hon. Treasurer : Mr. P. Winfield.
Committee : Messrs. Barry Russoff, B. J. White.
Co-opted Members : Messrs. S. Burge and M. Vaswani.

A visit to the steelworks of Messrs. Dawnays Ltd., at Welwyn Garden City has been arranged for the morning of Saturday, July 10th. Lunch will be provided ; the return fare from King's Cross is 5s. 6d. The number of visitors is limited to 35, and those wishing to participate should make early application to the Hon. Secretary.

A visit to London Airport has been arranged for the morning of Saturday, August 21st. A coach will leave the Institution at 8.45 a.m. The return fare is 7s. 0d. The number of visitors is limited to 35, and those wishing to participate should make application, enclosing the coach fare, to the Hon. Secretary, J. A. Pope, 53, Cranleigh Drive, Leigh-on-Sea, Essex.

Lancashire and Cheshire Branch

The following Hon. Officers and Committee Members have been elected for the session 1954-55 :—

Chairman : Mr. W. D. Blades.
Vice-Chairman : Mr. J. H. Morris.
Immediate Past Chairman : Professor J. A. L. Matheson.
Hon. Secretaries : Mr. A. S. Sinclair, 17, The Circuit, Cheadle Hulme, Cheshire ; Mr. M. D. Woods, 58, Spring Gardens, Salford, Lancs.
Hon. Assistant Secretary : Mr. J. S. Parr.
Commitee Members : Messrs. G. Greenlees, A. E. Wright, D. D. Matthews, W. Fitton, A. V. Booth, S. Gleaves, J. B. G. Martin, R. Gray, K. Norrey, W. Bates, H. J. Dowling, J. R. Bewick.
Hon. Auditors : Messrs. K. Norrey and J. R. Bewick.

Midland Counties Branch

Hon. Secretary : L. A. Firminger, A.M.I.Struct.E., 656, Chester Road, Erdington, Birmingham, 23.

Graduates' and Students' Section

Hon. Secretary : H. L. Bramwell, 139, Wood Lane, Handsworth Wood, Birmingham, 20.

Northern Counties Branch

The following Honorary Officers and Committee Members have been elected for the Session 1954-1955 :—

Chairman : Mr. W. G. Gentry.
Vice-Chairman : Mr. E. A. Parsons.
Immediate Past Chairman : Mr. T. H. Bryce.
Branch Hon. Secretary : Mr. O. Lithgow, 4, Stoneleigh Avenue, Acklam, Middlesbrough.

Tyne Centre Hon. Secretary : Mr. J. Whitten.

Branch Hon. Treasurer : Mr. L. Dobson.

Committee: Tees Centre—Messrs. E. G. Clark, T. Johnson, A. Burton, H. Rees, D. W. Portus, T. R. Tighe. Tyne Centre—Messrs. C. A. Harding, W. H. G. Durose, E. Atkinson, D. M. O'Herlihy, W. R. Garrett, H. W. MacKrill.

Hon. Auditors : Messrs. E. R. Fryer and D. W. Cooper.

NORTHERN IRELAND BRANCH

Hon. Secretary : A. H. K. Roberts, B.A., B.A.I., M.I.Struct.E., M.I.C.E.I., "Barbizon," 26, Dunlambert Park, Belfast.

SCOTTISH BRANCH

Hon. Secretary : G. Drysdale, A.M.I.Struct.E., "Niaroo," 33, Union Street, Motherwell, Lanarkshire.

SOUTH-WESTERN COUNTIES BRANCH

Hon. Secretaries : E. W. Howells, M.I.Struct.E., 10-12, Market Street, Torquay, Devon ; C. J. Woodrow, "Elstow," Hartley Park Villas, Tavistock Road, Plymouth, Devon.

WALES AND MONMOUTHSHIRE BRANCH

At the Annual General Meeting of the Branch, which was held at the South Wales Institute of Engineers, Cardiff, the following Honorary Officers and Committee members were elected for the Session 1954-55 :—

Chairman : Mr. F. V. M. Bell.

Senior Vice-Chairman : Mr. A. V. R. Hooker.

Junior Vice-Chairman : Mr. G. E. Cooper.

Hon. Secretary : Mr. K. J. Stewart.

Hon. Assistant Secretary for North Wales : Mr. S. C. Brown.

Hon. Assistant Secretary for Cardiff : Mr. W. D. Hollyman.

Hon. Auditors : Messrs. W. G. Spooner and E. O. Jones.

Committee : Messrs. G. H. Hodgson, W. A. Evans, Dr. A. A. Fordham, Colonel R. D. Heseltine, Messrs. F. M. Fordham, D. Manolopoulos, A. G. Thompson, E. O. Jones, J. W. Partridge, G. R. Brueton, E. R. Steward, G. W. Spooner, D. A. Lewis, J. E. Jenkins, J. L. Bannister and H. V. Morris.

Mr. G. R. Brueton, who has held the office of Hon. Secretary for the past two years, tendered his resignation, which was accepted with great regret and it was agreed to place on record the appreciation of the Branch of the valuable services rendered by Mr. Brueton during his term of office.

Hon. Secretary : K. J. Stewart, A.M.I.C.E., A.M.I. Struct.E., 15, Glanmor Road, Swansea, Glam.

WESTERN COUNTIES BRANCH

Hon. Secretary : E. Hughes, A.M.I.Struct.E., 23, Southdown Road, Westbury-on-Trym, Bristol, 9.

YORKSHIRE BRANCH

Hon. Secretary : E. Wrigley, A.M.I.Struct.E., 17, The Drive, Alwoodley, Leeds.

UNION OF SOUTH AFRICA BRANCH

Branch Hon. Secretary : A. E. Tait, B.Sc., A.M.I.C.E., A.M.I.Struct.E., P.O. Box 3306, Johannesburg. During week-days Mr. Tait can be contacted in the City Engineer's Department, Town Hall, Johannesburg. 'Phone : 34-1111, Ext. 257.

Natal Section Hon. Secretary : E. G. Bennett, A.M.I.Struct.E., c/o The Reinforcing Steel Co., Ltd., P.O. Box 49, Merebank, Durban.

Cape Section Hon. Secretary : R. Stubbs, M.I.Struct.E., The Reinforcing Steel Co., Ltd., P.O. Box 2692, Cape Town.

ADDITIONS TO THE LIBRARY

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Portal Frame Analysis by Moment Area Methods

By J. F. Horridge, A.M.I.Struct.E.

Summary

The method presented here is applicable only to portal frames with hinged base columns, but as about 80 per cent. of every-day portal design comes under this category, it must be agreed that this fact does not seriously affect its usefulness to the designer.

The method is a "true" one—that is, as "true" as Mohr's Moment Area Theorems—and does not require any approximations or trial-and-error calculations. It can be used for the solution of portal frames of any number of varying spans, and with deck beams (or rafters) and columns of varied inertia and elasticity.

Several examples are given, including a 4-bay structure with assymetrical loading.

Introduction

Consider a simple bent as in Fig. 1. The feet are hinged and free to deflect horizontally. From Mohr's Theorem we can say that the deflection at B relative to the tangent at A is equal to the moment of area of the bending moment diagram between A and B taken about B and divided by $E.I.$ Owing to the points A and B being in line, however, the tangent at A does not affect the calculations, and all moments of area are taken vertically about the base line $A.B.$

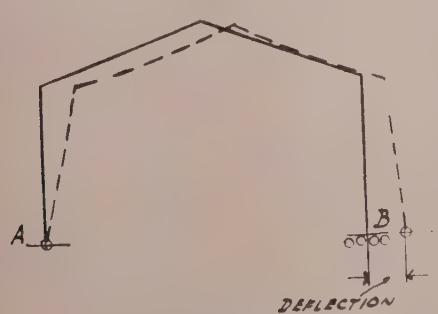


Fig. 1

The method is to draw two free bending moment diagrams—that is, if the feet were free to deflect horizontally—one for the loading and reactions, and one for the unknown horizontal reactions H. Knowing that there will in fact be no horizontal deflection, we add the moments of area for the two diagrams and equate to zero, thus finding H. The following example will illustrate the method fully.

EXAMPLE 1

A flat decked portal frame is shown in Fig. 2 with a side load of 5 tons. Bending moment diagrams in ft. tons are plotted first for the loading and known reactions, and second for the unknown horizontal reactions H, which we have assumed to be acting inwards. Bending moment diagrams are plotted on the tension side of members, therefore all diagrams on the outside of a frame will tend to deflect the feet inwards—the reverse for diagrams on the inside of a frame.

The equation used is :—

Outward deflection of feet = 0

Therefore :

Diagrams on inside of frame are positive.

Diagrams on outside of frame are negative.

Lever arms for moments of area are taken vertically from the base line to the point where the C.O.G. of the area intersects the actual frame outline. For simplicity, I and E have been taken as constant for this example, and are therefore omitted from the calculations.

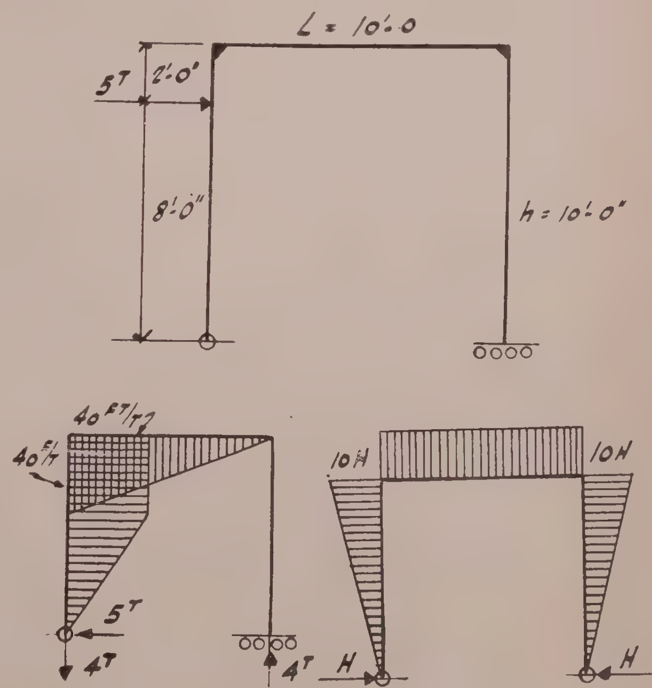


Fig. 2

Note the distribution of the horizontal load to the bases in the loading bending moment diagram. The left-hand column base takes the full reaction, the right-hand column base being free to deflect.

Taking moments of area about the base line of both diagrams, the outward deflection of the feet equals :

$$\left(40 \times \frac{8}{2} \times \frac{2 \times 8}{3}\right) + (40 \times 2 \times 9) + \left(40 \times \frac{10}{2} \times 10\right)$$

$$-2\left(10H \times \frac{10}{2} \times \frac{2}{3} \times 10\right) - (10H \times 10 \times 10) = 0$$

$$\therefore\ 3573 - 1666H = 0$$

$$\therefore\ H = +\ 2.15\text{T}$$

This positive sign indicates that our original assumption of the direction of H was correct, and the final bending moment diagram is shown in Fig. 3. It will be

seen that the outward deflection due to the unknown horizontal reaction H (i.e., for this case—$1666H$) is a constant for the frame. For any further conditions of loading, the free bending moment diagram only need be

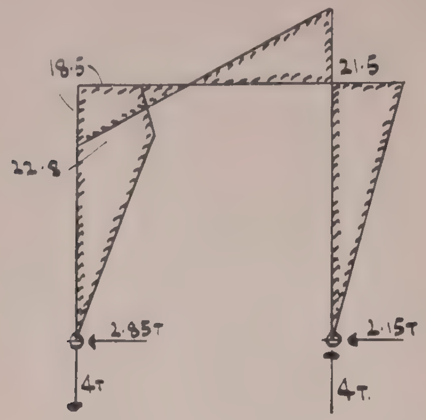


Fig. 3

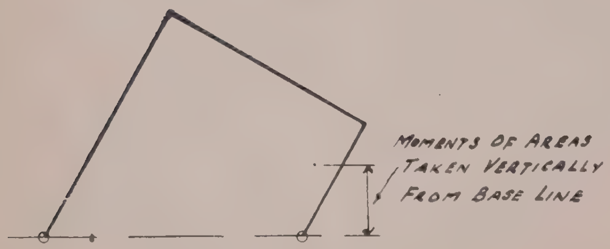


Fig. 4

drawn, and the moments of area of this diagram equated along with the H constant to O. This method can save a great amount of time and work in the analysis of frames subject to numerous conditions of loading.

Frames with unequal columns can be treated as in Fig. 4.

EXAMPLE 2

Consider now a typical two-hinged ridge type portal as shown in Fig. 5. The inertia of the rafters and columns have been taken as 2 and 1 respectively, and have now to be included in the calculations. E being constant is again omitted. This first fraction in each section is the number of areas/inertia of section.

Outward deflection of feet equals :

$$\left(\frac{2}{2}\times 108\times 39\times\frac{2}{3}\left[15+\frac{5\times 15}{8}\right]\right)-\left(\frac{2}{1}\times 15H\times\frac{15}{2}\times\frac{2\times 15}{3}\right)-\left(\frac{2}{2}\times 15H\times 39\times 22.5\right)$$

$$-\left(\frac{2}{2}\times 15H\times\frac{39}{2}\times 25\right)=0$$

$$\therefore\ 68445-22725H=0$$

$$\therefore\ H=3\text{T}.$$

The final bending moment is shown in Fig. 6.

Deflection Across Knees

The method can also be used to calculate the actual deflection across the knees of a portal. Taking the two bending moment diagrams in Fig. 5, draw a new "base line" at knee level, and take moments of area vertically about this line. Assuming the chosen design section for the rafters to be a 16 in. × 6 in. × 50 lb. R.S.J. ($I = 618$) and taking E as 13,000 tons per sq. in.

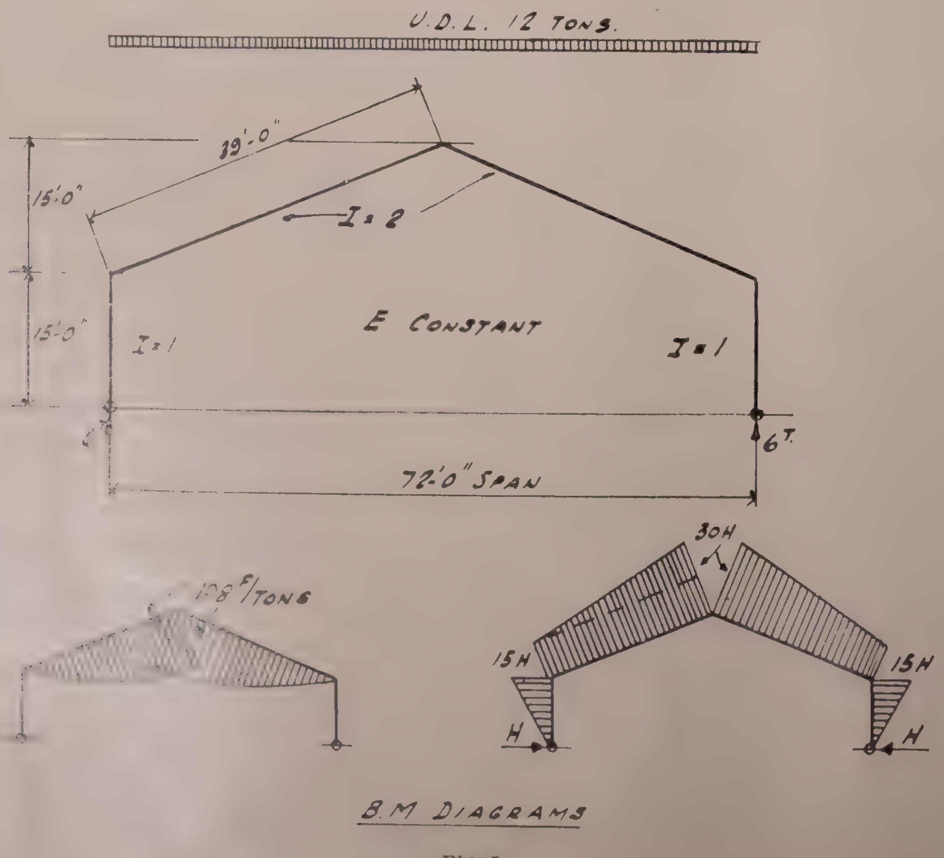


Fig. 5

Outward deflection across knees :

$$\left[\left(108 \times 39\times \frac{2}{3}\ \frac{(5 \times 15)}{8}\right)-(15 \times 3 \times 39\times 7{\cdot}5) - \left(15\times 3\times \frac{39}{2}\times 10\right)\right]\frac{2\times 12^3}{13{,}000 \times 618} = 1{\cdot}88 \text{ in.}$$

This calculation could, of course, have been simplified, but the figures have been presented in full for illustration.

Table I has been compiled giving formulæ for the outward deflections at feet of symmetrical two-hinged portal frames for common conditions of loading and moments applied over columns. These formulæ will be used where applicable in the following examples to simplify calculations.

diagrams must be taken into account along with the free bending moment diagrams.

Table 2 gives the continuity moments over columns due to H forces only for multi-span frames up to four bays—all bays being similar and with deck beams (or rafters) of constant I and E. These formulæ have been calculated by ordinary moment area methods and can, of course, be extended to any number of span desired.

Any error in the assumed directions of these horizontal forces will be shown finally in the calculations with a negative sign.

The following example of a 2-span flat decked portal is treated in detail to illustrate the method.

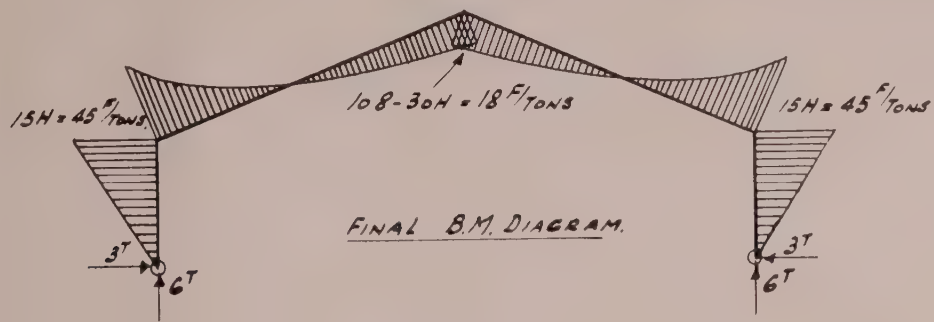


Fig. 6

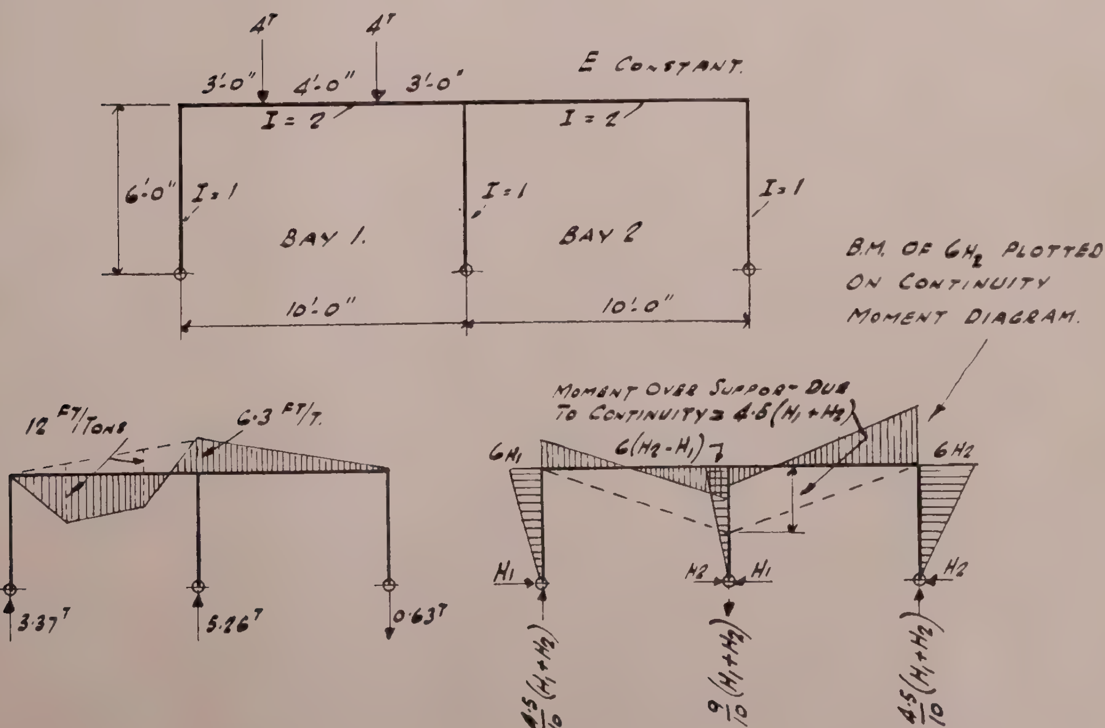


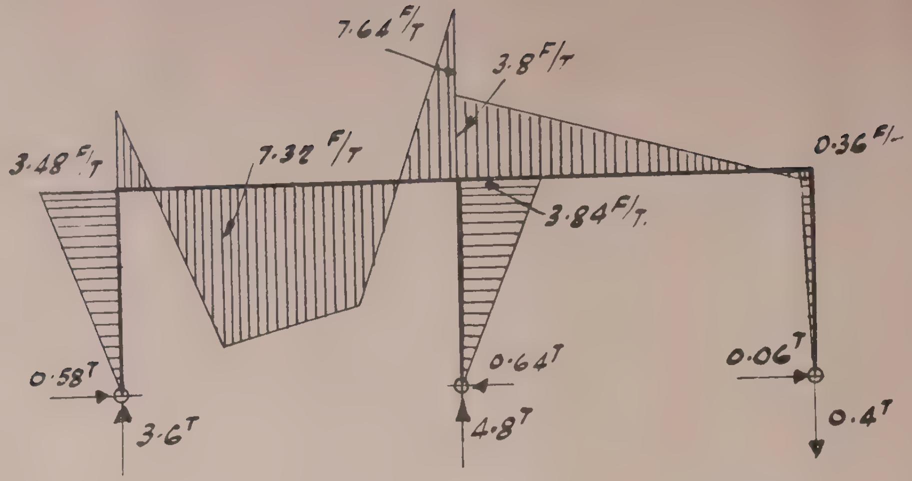
Fig. 7

Multi-Span Portal Frames

Mutli-span portal frames are treated in exactly the same manner as for single-span frames, each bay being treated separately and the results equated together. Care must be taken, however, of the continuity moments over the columns, and the area of these bending moment

EXAMPLE 3

Fig. 7 shows a loading diagram and bending moment diagrams of a 2-bay structure. It must be realised when plotting free bending moment diagrams that the columns, being free to deflect, have no effect on the deck beams at all, and the beam acts as simply supported, or if continuous over a column, free to rotate. The continuity moments over the columns are calculated by ordinary methods for continuous beams. Considering firstly Bay I.



FINAL B.M. DIAGRAM

Fig. 8

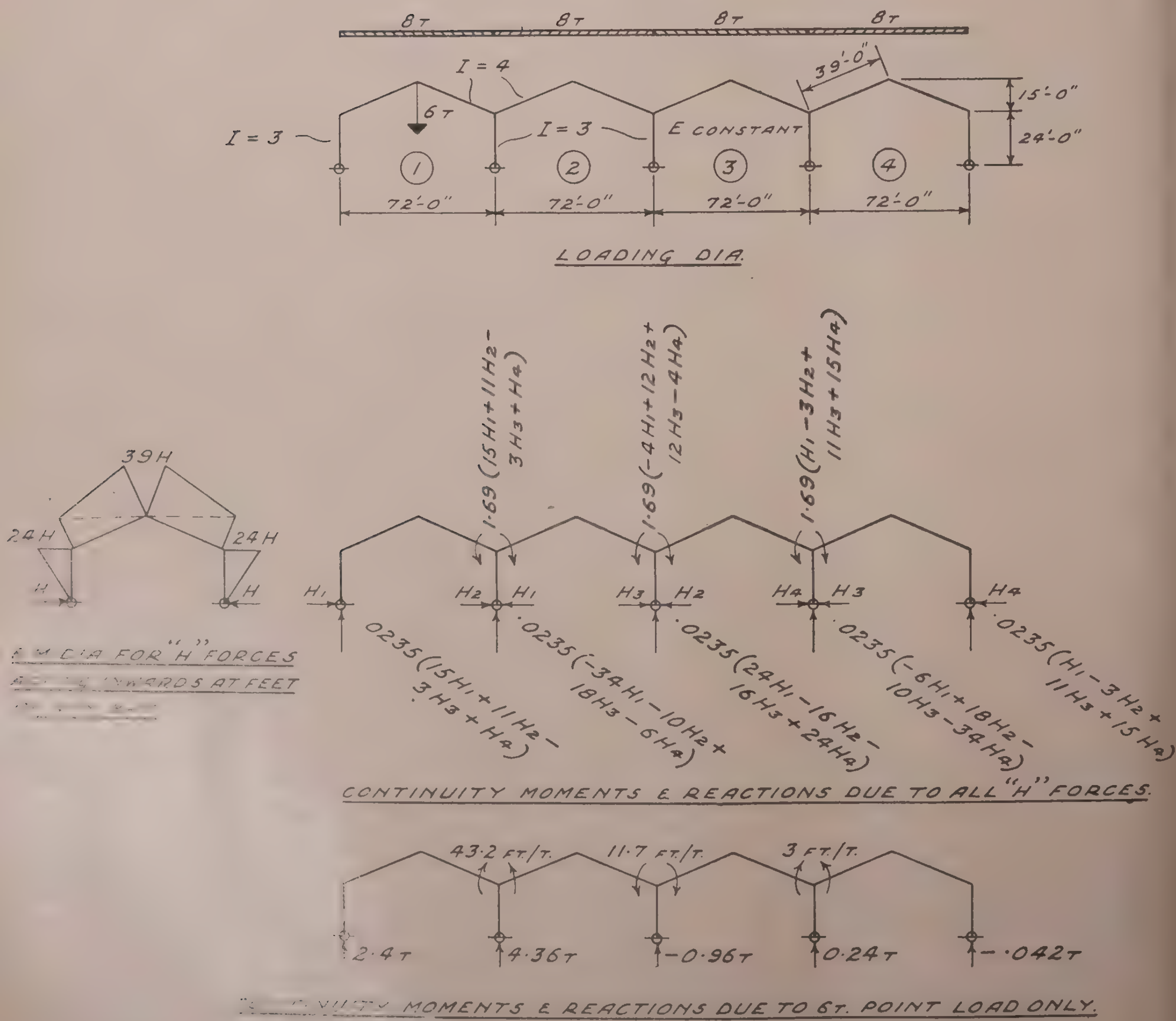


Fig. 9

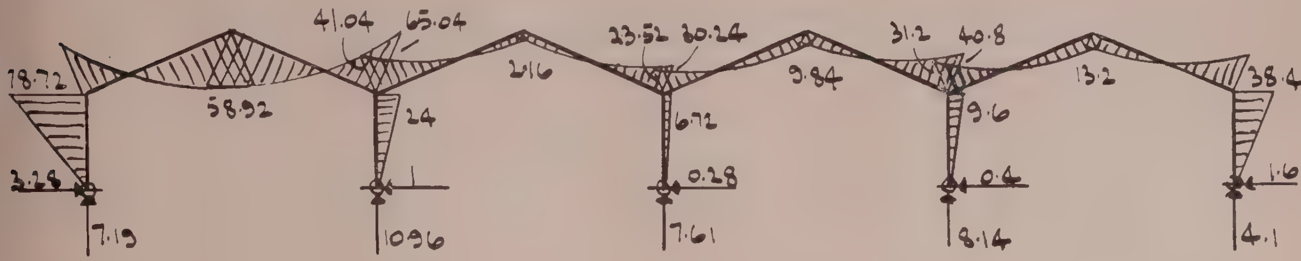
Outward deflection of feet due to dead loads only equals :

$$\left(\tfrac{1}{2}\times 12\times(4+3)\times 6\right)-\left(\tfrac{1}{2}\times 6.3\times\frac{10}{2}\times 6\right)=157.5$$

BAY 2

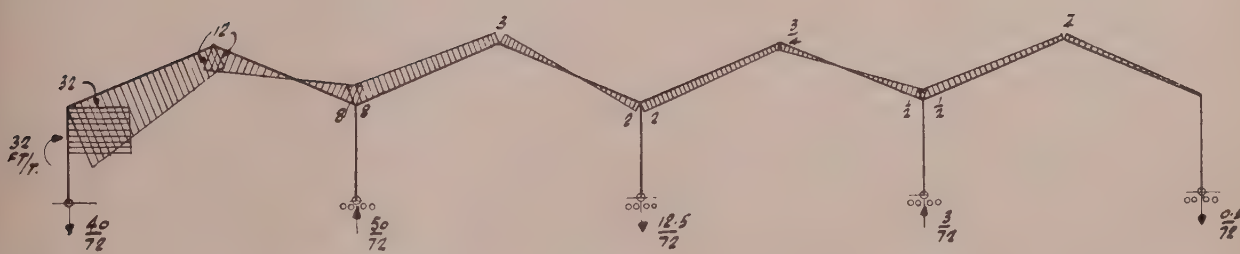
Outward deflection of feet due to dead loads only equals :

$$-\tfrac{1}{2}\times 6.3\times\frac{10}{2}\times 6=-94.5$$

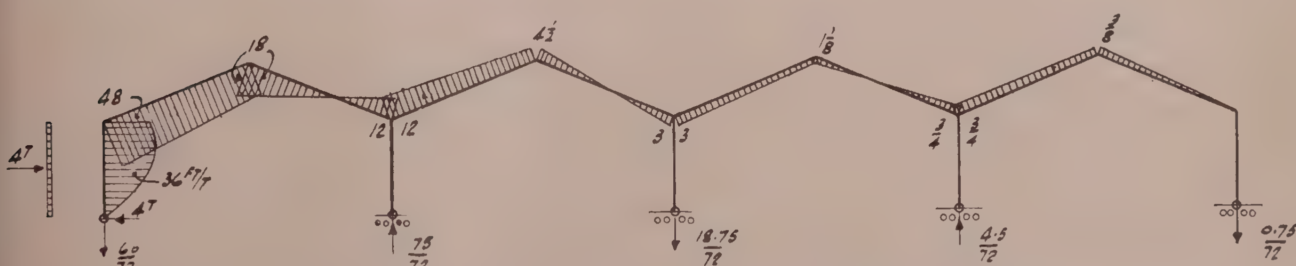


FINAL B.M. DIAGRAM.

Fig. 10



TYPICAL FREE B.M. DIAGRAM FOR ECCENTRIC BRACKET TO EXAMPLE 4



TYPICAL FREE B.M. DIAGRAM FOR SIDE U.D. LOAD TO EXAMPLE 4

Fig. 11

Outward deflection of feet due to H forces only equals :

$$-\left(\frac{1}{1}\times 6H_1\times\frac{6}{2}\times\frac{2}{3}\times 6\right)+\left(\frac{1}{1}\times 6(H_2-H_1)\times\frac{6}{2}\times\frac{2}{3}\times 6\right)-\left(\tfrac{1}{2}\times 6H_1\times 10\times 6\right)$$

$$+\left(\tfrac{1}{2}\times 4.5\,(H_1+H_2)\times\frac{10}{2}\times 6\right)$$

$$=-256.5H_1+139.5H_2$$

Summing all moments of area for Bay 1 :

$$157.5-256.5H_1+139.5H_2=0 \quad . \quad . \quad . \quad . \quad (1)$$

The two bays being symmetrical, it is obvious that the outward deflection of feet due to H forces will be similar to Bay 1, but with H_1 and H_2 reversed.

$$=139.5H_1-256.5H_2$$

Summing all moments of area for Bay 2 :

$$-94.5+139.5H_1-256.5H_2=0 \quad . \quad . \quad . \quad . \quad (2)$$

Solving the two equations 1 and 2 we have :

$H_1 = 0.58$ tons. $\quad H_2 = -.06$ tons.

H_2 acts in the opposite direction from that assumed. Horizontal and vertical reactions for the two bending moment diagrams are now added together and placed at the feet of the portal and the final bending moment diagram plotted as in Fig. 8.

4 Bay Portal Structure

Example 4

Consider now the portal structure as shown in Fig. 9. Besides the uniform distributed load of 8 tons per bay, a 6-ton point load is hung from the centre of Bay 1. The inertia of stanchions is assumed as 3, the rafters 4. E being constant is omitted from the calculations. The moments of area for the different sections of bending moment diagrams are treated separately for clearness, and each calculation is for the outward deflection of feet (abbreviated O.D.F.) due to that section of bending moment diagram only.

Dealing with each bay individually :

Bay 1

O.D.F. due to H_1 forces acting inwards at the feet, equals :

$$-\left(\frac{2}{3}\times 24H_1 \times \frac{24}{2}\times 16\right)-\left(\frac{2}{4}\times 39H_1 \times \frac{39}{2}\times 34\right)$$

$$-\left(\frac{2}{4}\times 24H_1 \times \frac{39}{2}\times 29\right) = -\ 22786.5H_1$$

O.D.F. due to H_2 forces acting outwards at foot of second column, equals :

$$\frac{1}{3}\times 24H_2 \times \frac{24}{2}\times 16 = 1536H_2$$

O.D.F. due to U.D.L., equals :

$$\frac{936}{14}(183.75) = 12285.$$

O.D.F. due to 6-ton point load simple bending moment diagram (using formula in Table 1), equals :

$$\frac{6\times 72\times 39}{4\times 4}(34) = 35802.$$

O.D.F. due to continuity moment over column from point load, equals :

$$-39 \times \frac{31.5}{4}\times 43.2 = -13268.$$

Summing all moments of area for Bay 1, equals :

$$34819-15012.4H_1+7237H_2-1555H_3+518H_4 = 0 \quad (1)$$

Bay 2

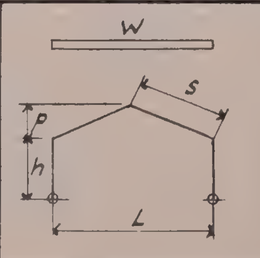
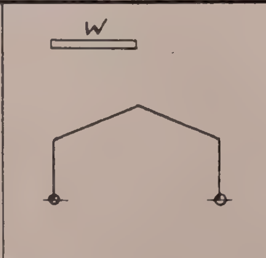
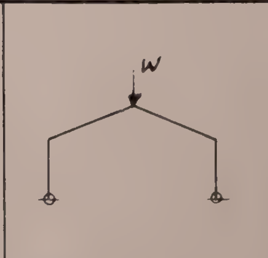
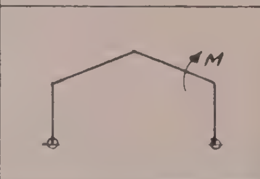
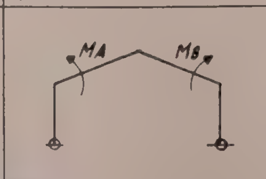
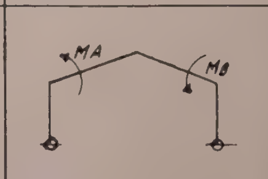
O.D.F. due to H_2 forces acting inwards at the feet, equals :

(As for Bay 1.) $-22786.5H_2$

O.D.F. due to H_1 and H_3 forces acting outwards at the feet, equals :

$$\frac{1}{3}\times 24\,(H_1+H_3)\,\frac{24}{2}\times 16 = 1536H_1+1536H_3$$

TABLE I

NOTE! I_1 = INERTIA OF RAFTER			
OUTWARD DEFLECTION AT FEET	$\frac{WLS}{6EI}\left(h+\frac{5}{8}P\right)$	$\frac{WLS}{6EI}\left(h+\frac{5}{8}P\right)$	$\frac{WLS}{4EI}\left(h+\frac{2}{3}P\right)$
			
OUTWARD DEFLECTION AT FEET	$\frac{-SM}{EI_1}\left(h+\frac{P}{2}\right)$	$\frac{-S}{EI_1}(M_A+M_B)\left(h+\frac{P}{2}\right)$	$\frac{-S}{EI_1}(M_A-M_B)\left(h+\frac{P}{2}\right)$

[illegible] tinuity moments over columns from [illegible] formula for moments over supports [illegible]

$$[illegible](15H_1+11H_2-3H_3+H_4)$$

$$[illegible]H_1+5701H_2 - 1554.8H_3+518.3H_4$$

Table 2 gives outward deflection for uniformly distributed loading.

O.D.F. due to continuity moments over columns from all H forces, equals :

$$39 \times \frac{31.5}{4}\times 1.69(11H_1+23H_2+9H_3 - 3H_4)$$

$$= 5701H_1+11920.3H_2+4664.5H_3 - 1554.8H_4$$

O.D.F. due to U.D.L., equals :

$$-\frac{936}{14}(5.25) = -351$$

O.D.F. due to continuity moments over columns from point load, equals :

$$-\frac{39}{4}(31.5)\ (31.5) = -9674$$

Summing all the moments of area for Bay 2, equals :

$$-10025+7237H_1-10866H_2+6200H_3-1555H_4 = 0 \quad . \quad . \quad . \quad (2)$$

BAY 3

Owing to the symmetry of the structure, the moments of area from all H forces for Bay 3 will be exactly opposite to Bay 2. Similarly for Bay 4 and Bay 1.

O.D.F. due to all H forces equals :

(Opposite to Bay 2.) $-1555H_1+6200H_2-10866H_3+7237H_4$

O.D.F. due to U.D.L., equals :

(As for Bay 2.) -351

O.D.F. due to continuity moments over columns from point load, equals :

$$-\frac{39}{4}(-8.7)\ (31.5) = 2672$$

Summing all the moments of area for Bay 3, equals :

$$2321-1555H_1+6200H_2-10866H_3+7237H_4 = 0\ .\ (3)$$

BAY 4

O.D.F. due to all H forces, equals :

(Opposite to Bay 1.) $+518H_1-1555H_2+7237H_3-15012.4H_4$

O.D.F. due to U.D.L., equals :

(As for Bay 1.) 12285

O.D.F. due to continuity moments over columns from point load, equals :

$$-\frac{39}{4}(3)\ (31.5) = -921$$

Summing all the moments of area for Bay 4 :

$$11364+518H_1-1555H_2+7237H_3-15012.4H_4 = 0 \quad . \quad . \quad . \quad (4)$$

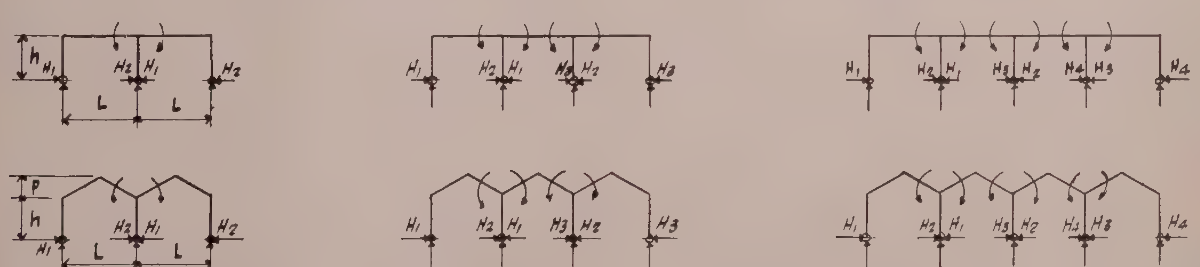
Solving the four equations 1, 2, 3 and 4, we find the following H values :

$H_1 = 3.28 \quad H_2 = 2.28 \quad H_3 = 2 \quad H_4 = 1.6$

TABLE 2 REACTIONS AND MOMENTS OVER SUPPORTS DUE TO 'H' FORCES ONLY

BAYS SYMMETRICAL AND OF EQUAL SPANS – DECK BEAMS (OR RAFTERS) OF CONSTANT E AND I

$K \begin{cases} \text{FOR FLAT DECK} = h \\ \text{FOR RIDGE TYPE} = h+\frac{p}{2} \end{cases}$ I = INERTIA OF BEAM OR RAFTER



	Two bays	Three bays	Four bays
MOMENTS OVER SUPPORTS	$\frac{3}{4}K(H_1+H_2)$	$\frac{K}{5}(4H_1+3H_2-H_3)$ $\frac{K}{5}(-H_1+3H_2+4H_3)$	$\frac{3K}{56}(15H_1+11H_2-3H_3+H_4)$ $\frac{3K}{56}(-4H_1+12H_2+12H_3-4H_4)$ $\frac{3K}{56}(H_1-3H_2+11H_3+15H_4)$
REACTIONS	$\frac{3K}{4L}(H_1+H_2)$ $\frac{3K}{4L}(-2H_1-2H_2)$ $\frac{3K}{4L}(H_1+H_2)$	$\frac{K}{5L}(4H_1+3H_2-H_3)$ $\frac{K}{5L}(-9H_1-3H_2+6H_3)$ $\frac{K}{5L}(6H_1-3H_2-9H_3)$ $\frac{K}{5L}(-H_1+3H_2+4H_3)$	$\frac{3K}{56L}(15H_1+11H_2-3H_3+H_4)$ $\frac{3K}{56L}(-34H_1-10H_2+18H_3-6H_4)$ $\frac{3K}{56L}(24H_1-16H_2-16H_3+24H_4)$ $\frac{3K}{56L}(-6H_1+18H_2-10H_3-34H_4)$ $\frac{3K}{56L}(H_1-3H_2+11H_3+15H_4)$

We can now evaluate the vertical reactions and plot the final bending moment diagram, which is shown in Fig. 10. The outward deflection at eaves of the different bays could quite easily be calculated by drawing a new " base line " across the knees and taking moments of area vertically above this line for each bay separately. It would, in fact, be found that Bays 2 and 3 have a slight inward deflection, being shown in the answer with a negative sign.

For illustration, two further free bending moment diagrams have been plotted for different conditions of loading to the above structure (Fig. 11), one for an

eccentric bracket to the left-hand column, and one for a side wind load. By using the different H constants already calculated for each Bay (i.e., Bay 1 equals $-15012.4H_1+7237H_2-1555H_3+518H_4$), and summing and equating to zero with the moments of area for these free bending moment diagrams, the H forces for the different loading conditions can be found with comparative ease and speed, but lack of space does not permit further investigation here.

Conclusions

It is realised that all uses of the " deflection method " have not nearly been expounded above. For instance, portal frames with a known amount of " side slip " in the bases could be analysed by equating the moments of area to the known deflection instead of zero. Also frames with ties across the knees or feet could be dealt with in a somewhat similar manner to Castigliano's methods.

In summing, it appears that the two main uses of the method are,

1. A neat, easy to understand, rapid, every-day method of analysis for single-span two-hinged frames under several different conditions of loading.

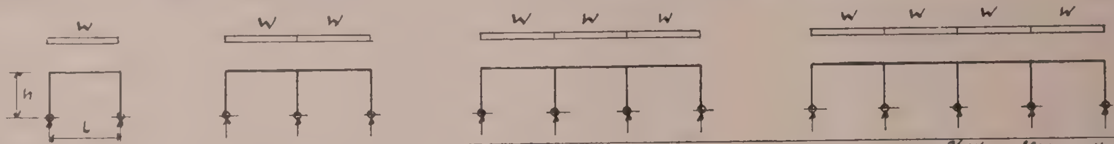
2. A simple method of calculating outward deflection across knees of assymetrically shaped frames.

TABLE 3

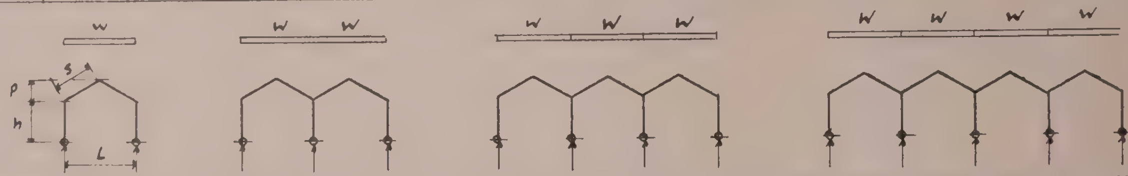
REACTIONS AND OUTWARD DEFLECTION OF FEET DUE TO U.D. LOADING.

BAYS SYMMETRICAL & OF EQUAL SPANS – DECK BEAMS (OR RAFTERS) OF CONSTANT E AND I

$K = \frac{WL^2h}{12EI}$ $V = \frac{WLS}{6EI}$ I = INERTIA OF BEAM OR RAFTERS



REACTIONS	$\frac{W}{2}$ $\frac{W}{2}$	$\frac{3W}{8}$ $\frac{10W}{8}$ $\frac{3W}{8}$	$\frac{4W}{10}$ $\frac{11W}{10}$ $\frac{11W}{10}$ $\frac{4W}{10}$	$\frac{11W}{28}$ $\frac{32W}{28}$ $\frac{26W}{28}$ $\frac{32W}{28}$ $\frac{11W}{28}$
OUTWARD Δ AT FEET OF BAY	K	$\frac{K}{4}$ $\frac{K}{4}$	$\frac{6K}{5}$ $\frac{-K}{5}$ $\frac{6K}{5}$	$\frac{5K}{14}$ $\frac{-K}{14}$ $\frac{-K}{14}$ $\frac{5K}{14}$



REACTIONS	$\frac{W}{2}$ $\frac{W}{2}$	$\frac{3W}{8}$ $\frac{10W}{8}$ $\frac{3W}{8}$	$\frac{4W}{10}$ $\frac{11W}{10}$ $\frac{11W}{10}$ $\frac{4W}{10}$	$\frac{11W}{28}$ $\frac{32W}{28}$ $\frac{26W}{28}$ $\frac{32W}{28}$ $\frac{11W}{28}$
OUTWARD Δ AT FEET OF BAY	$V(h+\frac{5}{8}P)$	$\frac{V}{4}(h+P)$ $\frac{V}{4}(h+P)$	$\frac{V}{5}(2h+\frac{13}{8}P)$ $\frac{-V}{5}(h-\frac{P}{8})$ $\frac{V}{5}(2h+\frac{13}{8}P)$	$\frac{V}{14}(5h+\frac{17}{4}P)$ $\frac{-V}{14}(h-\frac{5}{4}P)$ $\frac{-V}{14}(h-\frac{5}{4}P)$ $\frac{V}{14}(5h+\frac{17}{4}P)$

The author considers the above methods to be entirely original, and would welcome suggestions for improvement or further adaption from interested members.

Acknowledgements

The author wishes to thank Mr. J. Crane for his valuable assistance in the preparations of the sketches, and Mr. F. Bailey, A.M.I.Struct.E., for helpful criticisms and suggestions in the preparation of the draft.

Discussion

The Literature Committee would be glad to consider the publication of correspondence in connection with the above paper. Communications on this subject intended for publication should be received by November 1st, 1954.

Book Reviews

The World's Greatest Bridges, by H. Shirley Smith. (London : Phœnix House, Ltd., 1953.) 180 + xi pp., $8\frac{1}{2}$ in. × $5\frac{1}{2}$ in. Price 15s.

This book is of interest alike to the engineer, the student and the non-technical general reader. The historical aspect of bridge building and design is very adequately dealt with, thus giving added value to the book as a reference.

Written in very readable non-technical language, a very technical subject is dealt with in a manner which neither bores nor mystifies the layman. Fundamentals are explained in an interesting and simple manner and salient points in design and construction of the world's most famous bridges are given.

The work is well illustrated by excellent photographs and sketches and is provided with an exhaustive bibliography.

R. J. V.

Journal of the American Concrete Institute, 20-year Index, 1929-1949. (Michigan : A.C.I., 1950.) 256 pp., 9 in. × 6 in. $2.00.

This 20-year Index covers the period November, 1929, to June, 1949. It is divided into three main sections, index of subjects, authors and titles ; sources of equipment, materials and services ; and synopses of papers and reports, " Job Problems and Practice " items, and " Letters from Readers " items.

Design and Construction of Welded Portal Frame Warehouse Building Designed by the Plastic Method*

Discussion on the Paper by Mr. E. J. Callard, B.A., M.I.Mech.E.

The CHAIRMAN welcomed the visitors to the meeting and introduced the author.

Mr. CALLARD, in presenting his paper, commented that perhaps he was present on slightly false grounds because he could not claim to be an experienced structural engineer with full knowledge of the plastic theory. He had, however, been attracted by the advantages of the plastic method applied to a building of the type described, which was perhaps the simplest possible application of the plastic design theory to a steel-framed building.

The question of foundation design had been sketchily dealt with in the paper and it was perhaps of particular interest to consider the conditions for which the foundations supporting a structure of this type must be designed. The building foundations had been designed to withstand an overturning moment equal to the plastic moment of resistance of the section of the frame and not for the calculated moment arising from the fully factored loads. He felt it was of particular interest to discuss the criteria for foundation design and, in particular, whether in any frame in which a hinge does not occur first near to the stanchion base it is still necessary to design the foundation to resist the full plastic moment of the section. For a simple building such as was described in the paper, the question was largely of academic interest but in more complicated structures the cost of the foundations would be considerably changed by the need to design for the full plastic moment.

The paper described the method of construction employed in this particular case, but of course the method decided upon would vary according to the size of the structure, the economic circumstances, site access and available equipment.

Finally, he wished to correct an error in the printed version of the paper on page 32, immediately below Fig. 3. The bending moment equations should read :

At A, assuming a positive moment,
$$-M_1+Mx = -54.07$$
At B, for a negative moment,
$$-(M_1-15H)-Mx = -54.07$$
At C, for a positive moment,
$$-(M_1-27H)+Mx = 0$$

Discussion

The PRESIDENT proposed a most hearty vote of thanks to Mr. Callard for his paper and for the interesting manner in which he had introduced it.

Mr. S. VAUGHAN (Vice-President) first expressed his great appreciation of Mr. Callard's courage in having a building constructed in accordance with the recently developed plastic design method, and then in following this up by presenting a paper so ably to the Institution to show exactly what he had done.

All who had not had the opportunity of designing a structure in accordance with the plastic method would be impressed at once by the extreme simplicity of that method of design as compared with the normal elastic method. He supposed most structural engineers had waded through the long, laborious and tedious calculations which were necessary to solve a fixed-base pitched roof portal on the elastic basis ; the plastic basis offered an extremely simple alternative and logical method of design and, moreover, one which gave economy in the use of steel ; though he suggested and hoped to show that the economy effected was probably not so great as one might gather from the paper. However, there was no doubt whatever that a useful and valuable economy could be effected. In addition, the factor of simplicity appealed to him as he felt it must to all who were concerned with the design of structures of that kind.

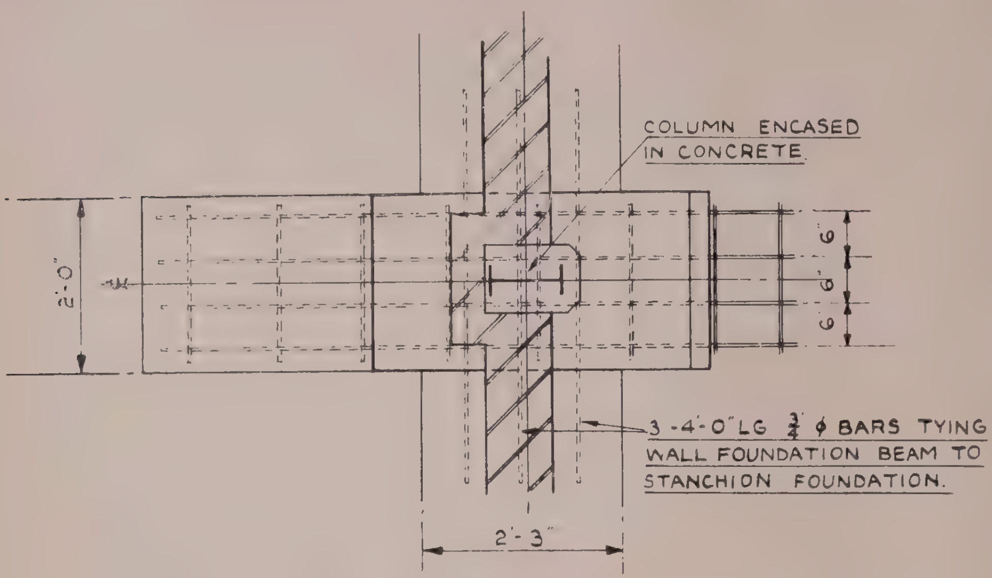
Having said how very much he appreciated and enjoyed the paper, Mr. Vaughan put forward a few tentative critical comments. He asked whether the lecturer had considered the alternative of using a pin-ended frame instead of a fixed-ended frame, and if so, why he had selected the fixed-ended design. Mr. Vaughan had worked out the case for the pin-jointed frame of the same dimensions as for the fixed-ended frame and loaded in the same way, and had found that although (as one would expect) the design moment was a little greater, nevertheless a 10 in. $\times$ $4\frac{1}{2}$ in. joist would still suffice. Had the author in fact investigated that possibility and did he agree that, using hinged ends for the bases of the stanchions, the size required would still be 10 in. $\times$ $4\frac{1}{2}$ in. ?

If the author did agree, Mr. Vaughan suggested that the foundations could have been constructed very much more economically because, instead of having to design the foundations for relatively small vertical load combined with a very large moment, necessitating a base of considerable size, he would merely have had to design them for a very small vertical load combined with a very small horizontal force at base level, and zero fixing moment, and a very considerable saving would have been effected in the foundations with no increase of cost whatever on the frame itself.

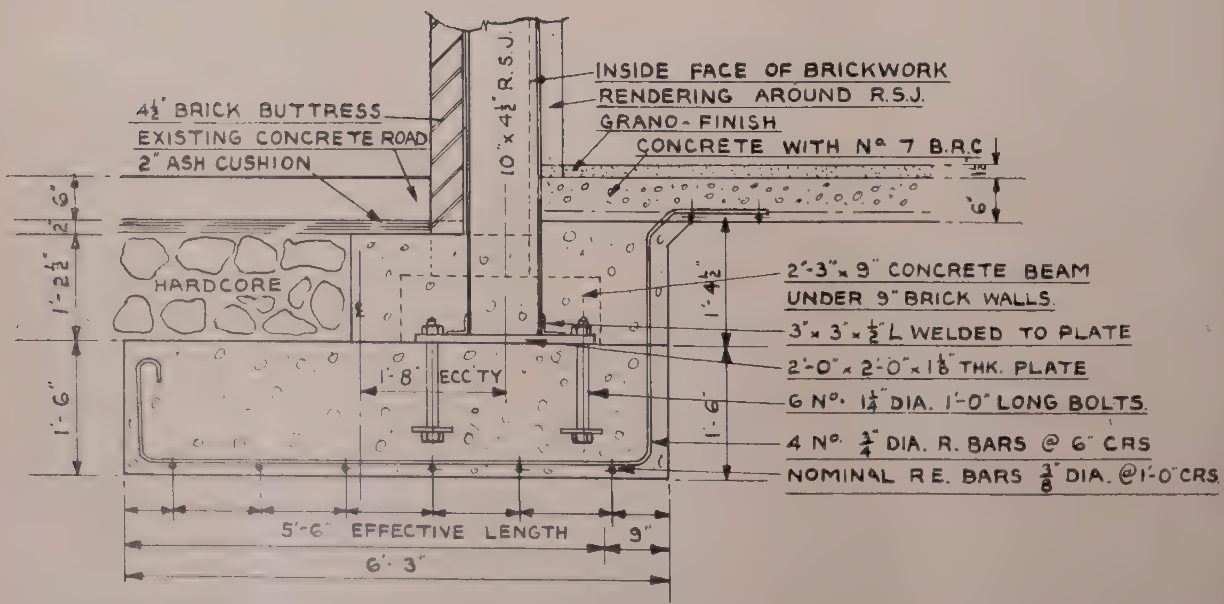
Coming to his second point, relating to the foundation design, he said the author had remarked that the presentation of that design was somewhat sketchy ; so that one could be excused for agreeing with him. He would have appreciated a drawing and a calculation for the foundations as designed. On page 34 the figures were given for the basis on which the foundations were designed. Whilst Mr. Vaughan agreed with the decision that it would be wise to design those foundations for the full plastic moment of the actual sections used, so as to

*Read before a Meeting of the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1., on Thursday, 28th January, 1954. The President, Lt.-Colonel R. F. Galbraith, M.C., B.Sc. (Eng.), M.I.Struct.E., A.M.I.C.E., in the chair. Published in THE STRUCTURAL ENGINEER, Vol. XXXII, No. 1, pp. 30-37.

give the same factor of safety on the foundations as on the frame, he could not make out where the author had obtained his figure of 6 tons axial loading. According to Mr. Vaughan's working, it seemed possible to have a smaller loading combined with the full moment. If he were right, the foundations would require to be even bigger, to provide the necessary dead weight and designed as a pin-ended frame, there would have been no fixing moment tending to cause tilting of the base—and moreover if any tilting were to occur this would in no way affect the design moments. Mr. Vaughan suggested, therefore, that for this reason, as well as for the reason already mentioned of overall economy, it might have been better, at any rate, in this particular



PLAN OF FOUNDATION



SECTION THRO' FOUNDATION FOR PORTAL FRAME

SCALE ½" = 1 FOOT

stabilising moment. Moreover, if the frame were designed as fixed-ended, it was vital that the foundations should be so designed and of such size that they did not allow any tilting whatever of the base under the action of the load ; any tilting in the direction of the moment would obviously affect adversely the design of the frame itself, and the moments in the frame would exceed those determined by the calculations. Had the frame been case, to have used a pin-ended design. He was sorry that the author had compared the plastic design with fixed bases against an elastic design with pinned bases ; if we really wanted to make an honest comparison between two methods of design we must take identical conditions, for otherwise the figures did not convey very much. He would agree that probably the difference on that score was not very great, but he still considered

that the conditions should have been identical, and the comparison should therefore have been between fixed-ended frames designed by the plastic and elastic methods respectively.

It seemed to him that the alternative size which was given for the elastic design, of 15 in. × 5 in. × 42 lb. per ft., was surprisingly heavy, but no calculations were given in the paper to explain why such a heavy section was needed. From his experience in other cases he had generally found that the saving of steel was much more of the order of 20 per cent. by using the plastic method as compared with the elastic method of design, whereas in the present case by the plastic design method the author could practically have used a 9 in. × 4 in. × 18 lb. per ft. joist, and by the elastic design he had arrived at 15 in. × 5 in. × 42 lb. per ft., the latter being more than twice the weight of the former. That had made Mr. Vaughan suspicious; he had felt that there was something which he did not understand or that possibly there was a slip in the calculations for the 15 × 5 R.S.J. He had therefore made independent calculations based on the elastic method using pinned joints, so as to keep in line with what the author had done. Subject to his arithmetic being right, he had arrived at a size of 10 in. × 5 in. × 30 lb. per ft. by the elastic method of design, showing a saving for the plastic design method of about 17 per cent.

He would be delighted if either the author, or perhaps later contributors to the discussion could check and verify that a 10 in. × 5 in. R.S.J. would suffice, so that we should all know the order of the economy which could be attained. It was, he believed, true that a plastic design would always give an economy over an elastic design. But it was important that any published figures comparing the cost of plastic with elastic designs should be accurate. We should be doing a dis-service to the plastic method of design (which was one to which he was very much attracted for simple frames of the kind discussed), if by some slip we should over-emphasise the savings which could be effected.

Mr. Callard said it was true, as he had stated at the beginning of the paper, that there were many things which he and his colleagues might have gone into at the time and which in fact they did not consider fully enough, as they now appreciated. The design and construction of the building was strictly a commercial job and in no way a research project, so that it was more important to complete a building economically designed than to spend undue time and effort arriving at a design which would use the minimum possible quantity of steel.

On the question of pin-jointed *versus* fixed-ended bases, he did not think they had worked out a full design on the basis of pinned joints. They had assumed that a fixed base portal would require a lighter section, and he believed it did for the same load factor. But as regards the foundation, which he considered was a most important and interesting consideration, they had already burned their boats on the fixed base portal before they had fully realised how much there was in the foundation consideration; had they worked out the case of the pin-jointed frame together with the foundations necessary by both the plastic and the elastic methods they might have been able to agree with Mr. Vaughan; but, not having worked it out, he could not say.

He could not check at that stage the axial loading of 6 tons; so that he was afraid he must leave that until later.

Concerning the question of the economy that was achieved he pleaded very largely guilty. He had claimed that he would achieve the economy stated when he compared the plastically designed fixed base portal as erected with the elastically designed pin-jointed frame, which was a preliminary design. In not having compared the two on a strictly fair and comparable basis he had perhaps been misleading and he felt there was a good deal in what Mr. Vaughan had said. Although it was true that $34\frac{1}{2}$ per cent. less steel had been used than would have been had they proceeded with the preliminary design, it should not necessarily be read that the $34\frac{1}{2}$ per cent. economy was due merely to the use of the plastic method, although undoubtedly there had been an economy arising from its use.

Dr. Jacques Heyman (Engineering Laboratories, Cambridge University) said he had been asked by Professor Baker to say how sorry he was that he was unable to attend the meeting; he had to attend a University Board meeting and could not get away in time.

The paper, Dr. Heyman continued, illustrated the economies which could be achieved by the plastic method of design, as well as the simplicity and rationality of that method; the author's results and claims resulting from his application of the plastic theory were confirmed by the experience at Cambridge in the design of other structures.

Discussing some of the points which seemed to crop up in the design of the sort of structure discussed in the paper, he began with the purlins. Mr. Callard had said that the effect on purlin weight of designing them by the plastic method was not investigated. At Cambridge it was found that the use of joist sections had been extremely economical, especially if the purlins were designed as fully continuous members. He had worked out that by using 3 in. × $1\frac{1}{2}$ in. × 4 lb. per ft. steel joists the saving effected was about 5 tons or 40 per cent. of the steel used in the purlins. That saving could be achieved only at the expense of making the purlins fully continuous, but there was the added complication that the purlins tended to be weak about their minor axis and some sort of stabilising was necessary, similar to that illustrated in Fig. 25 of the President's Address (The Structural Engineer, Vol. XXXII, No. 1), leading the purlins up to a strong ridge member with sag rods. Using the 3 in. × $1\frac{1}{2}$ in. joist, the load factor for an internal span was still very high, more than 3, and for an end span (propped cantilever), the load factor was still 2.4. The use of joist purlins either saved material in the purlins themselves or permitted an increase of the frame spacing. For example, there was a load factor of 2.4 for the purlins at a spacing of 15 ft.; the spacing could be increased to 18 ft. and the load factor would still remain above 2. Alternatively, using a 4 in. × $1\frac{3}{4}$ in. joist, the spacing could be increased to 29 ft., and the load factor would be 2. If the frame spacing were increased, heavier sections could be used and would prove more economical, since the moment of resistance per lb. of material increases as the section is increased. At Cambridge they had developed a very rough working rule, which was departed from in many cases. If the purlin weight were roughly equal to the frame weight for a structure of the type under discussion it was reckoned that the weight of material used would be a minimum. But that sort of rule was upset by other factors, e.g., larger spacing led to fewer foundations and fewer joints, which again showed the advantage of larger frame spacing.

Professor Baker, in a paper to the Institution of Structural Engineers in 1949 on "The Design of Steel Frames," had dealt with the effect of haunching on the main frames. We could haunch in two places, i.e., at the apex or at the eaves, and two effects were achieved

by haunching. First there was a strength effect. If we haunched at the apex, no strength gain was achieved, as was indicated in Fig. 3 of Mr. Callard's paper ; the plastic moment developed at the second purlin point from the apex and it was of no use putting extra material into the apex itself. On the other hand, if we haunched at the eaves, where the bending moment diagram was peaky, the prevention of the formation of the maximum moment at the knee by the use of the haunch would be beneficial. Secondly, haunching at the apex would stiffen up the frame because there was a region of almost constant moment at the apex and, if one were worried by deflections, that was a good place at which to put in some haunching.

Dealing with foundations, Dr. Heyman agreed with Mr. Vaughan that the use of a 10 in. $\times$ $4\frac{1}{2}$ in. joist was perfectly adequate for a pinned base portal. He had worked out the load factor at 1.83, as compared with 2.26 for the fixed base design. Looking at it in another way, he had worked out the moment for the pinned base at 19.5 tons ft., instead of the 15.8 tons ft. shown in Fig. 3.

The use of fixed feet for portals of that type was very tempting from the point of view of economy. One tried to use the material as well as possible and obviously there was a gain by fixing the feet, but that gain might be small and might be offset by the extra expense of providing adequate foundations.

Mr. Callard, in thanking Dr. Heyman for his remarks, said there were many things which he and his colleagues did not go into in the effort to arrive at the most economic structure, and they were conscious that they might have done a lot better.

Mr. R. J. Fowler, (Associate-Member), said that, plastic theory or elastic theory, the consideration of the welding details and method of erection should be the same ; but perhaps he was wrong.

It appeared to him that the method of erection used was a rather expensive one for such a structure. The arrangement of jigging, and so on, meant that either quite a considerable number of jigs must have been made, or alternatively the structure must have gone up slowly if only one set of jigs were available, and that seemed to point to a rather expensive erection programme.

A figure which enabled one to get one's teeth into the problem of the cost of erection of the job was that of the welding time, which was $27\frac{1}{2}$ man-days, in spite of the fact that frequent checks were made on dimensional accuracy. It amounted to about £6 per ton just for carrying out the welding, apart from erection labour, etc., and that appeared to him to be rather a high figure for such a light structure which would offset any savings effected by the use of the plastic theory. By that he meant that, had the structure been designed on the elastic basis and had rather quicker erection arrangements been made, the erection might have cost less.

The point had been made that two particular welding electrodes were used because they gave great penetration into the root of the weld. He considered that to be rather strange, because there were hosts of electrodes on the market which had been approved and which met British Standard requirements for welding of that nature, and he did not understand why it was that two makes of electrodes could be picked out as being superior to all others for the purpose of erecting the structure. He asked for a little more detail concerning the reasons for the choice.

Again, one had little difficulty in appreciating the description of the welds at the eaves and apex joints. It was stated that a certain arrangement would obviate the need for butt welds, and he did not understand that because the butt weld was the most economical weld that one could use to attain the full strength of the section, and square groove butt welds had in fact been used.

Coming back to the erection problem, he said that in welding one of the rafter frames, the whole frame, according to the method used, was turned completely over through one revolution, whereas it could have been dealt with just as well, with the same joint detail, by setting it out on the ground, tying it by tack welding a length of purlin to it to form an "A" frame, completing the butt welds by welding in the normal position, one on either side, balancing the welding to maintain the shape of the frame, lifting it so that the apex was (say) 6 ft. above the ground, and finishing the remaining fillet welding in the upside-down position.

Finally, Mr. Fowler complimented the author on a very valuable paper, and one which was at the same time courageous.

Mr. Callard, dealing with the question of cost, said that his Division had contracted the job at a fixed price and all he could say was that the price paid was less than he would have paid had the structure been designed by elastic methods. He did not say that by adopting other methods the erection cost could not have been cut down and he felt that, as a result of the experience gained, they would considerably reduce the erection time were they to proceed now with a similar job. He believed a good deal of the welding time taken was due to their wish to make quite certain that the welding was good, and partly resulted from the careful and frequent inspections made of each run of weld. It could be argued that this was unnecessary and that the welding time could have been considerably reduced. He did not think it was fair to say that because the stated number of man hours was devoted to the welding of this structure a welded frame, designed by the plastic method, is expensive to erect and he was very willing to admit that there might be ways of simplifying and cheapening the work.

It was not his intention at all to suggest that the electrodes used were chosen as being superior to all others. They had selected them following consultations with their metallurgical colleagues and some of their friends, but it might well be that there were many other electrodes which would have done an equally good job. They were particularly conscious that the positions of the welds in the frame were such that they would be subjected to high bending moments, and hence high stresses, and they were anxious to achieve a weld deposit which not only gave a joint of adequate strength but deposited metal which was at least as ductile as the parent section.

Mr. G. A. Jones, discussing the point raised by Mr. Vaughan and Dr. Heyman with regard to the use of pin-jointed rather than fixed-ended bases, said the comparative economy of the two would depend largely, of course, on the nature of the ground in which the foundations were laid. But the main consideration from his point of view would be the nature of the walls, and in the building discussed in the paper they were of brick. If we had a frame with pinned bases the lateral deflection would be unduly high to be combined with brickwork not wholly carried by the steel frame. That is to say the lateral loads would tend to be carried by the brickwork, putting a high stress on the mortar joint, and in his opinion the fixed base frame was the obvious choice in this case. If the walls had been sheeted, possibly the answer would have been the

pinned base, especially if the subsoil on which the foundations were placed was poor.

Concerning the design of purlins, there again he supposed there would be very much higher deflection than usual which would certainly become rather alarming to the plant engineers to whom the building was handed over eventually ; he had suspicion that the purlins would sag rather under heavy snow load.

Coming to the foundations, it would seem reasonable to him that if these were designed on the basis of a bearing pressure between steel and concrete of say 20 tons per sq. foot, they should be designed purely and simply to cope with the moment which was liable to be produced under working conditions at the foot of the column. On the other hand, if the foundations were to be designed to cope with the failure load, one would have to use the full crushing strength of the concrete and the ultimate strength of the clay with the proviso that in such circumstances the load factor would probably have to be increased for the foundation because, of course, the quality of the concrete is not so predictable as that of steel, nor is that of the ground on which the concrete is founded.

In conclusion, Mr. Jones said how very much he had enjoyed reading the paper and hearing Mr. Callard's presentation of it. Although at Billingham they were concerned also with corrosion, which brought its own problems, there was no doubt that the method of design discussed in the paper was the most rational basis for designing steel structure that had been evolved of recent years.

Mr. J. P. M. Bell (Associate-Member), on the question of whether a frame foundation should be designed for full plastic moment or working load moment, said that it was possible to visualise a frame of such shape that plastic hinges could develop at the bases at or very near working load and still be a long way from complete collapse. Under this condition the base should obviously be designed for full moment. At the other extreme a frame could develop base hinges last of all. Under this condition it would appear wrong to design the base for full plastic moment.

The theoretical ground pressure assumed in normal design gives a factor of safety of at least 2, and we seemed in this second case to be adding a double factor if we designed for full plastic moment. In the case of light buildings the question was perhaps a little academic but in the case of a very heavy building it could matter very much and could detract from the economy of the method. There was of course an intermediate case, rather indeterminate, where it would seem advisable to design for full plastic moment for safety.

However, he would like to hear what the Cambridge representative had to say on the matter.

Dr. Heyman, who was invited to comment on the point, said he quite agreed that the problem of foundation design was a very vexed one. The matter was under investigation at Cambridge.

Mr. Callard added that Mr. Bell and himself had discussed the matter and were very anxious to learn more about it, because it seemed to be an important technical point, and in connection with the more complicated structures it could be important economically. At the moment he would say that if his organisation were designing their building again they would provide a foundation to take the full plastic moment for the frame.

Mr. B. I. Clark (Associate-Member) said the problem involved the consideration not only of the strength of the column, but also of the foundation. If there is rotation of the foundation in a soil, then the fixed moment at the base would be relieved and, if the rotation of the base was sufficient, the base of the column would eventually become pin-jointed.

If we did not design a suitable foundation to resist the full moment of the section without any rotation and we then collapse the frame, a hinge would be developed at that point. Therefore, the foundations of such frames have to be rigid enough to resist any possible rotation, as otherwise, the frame becomes pin-jointed at the base.

When designing frames requiring complete rigidity at the base, one must give careful consideration to the foundation conditions and to the possibility of attaining and maintaining the conditions upon which the design was based.

Mr. Callard did not feel that he could add much in that connection because he just did not know. However he considered we must design a foundation which would fully withstand the turning moment, certainly under working load and probably under the load which produced a plastic hinge at the column base. It seemed that there could be cases where we might be allowed to design for an overturning moment less than the full plastic moment of the stanchions ; that was the crux of the matter, whether or not that was permissible. If there were frames of that type which were increasingly loaded and, considered elastically, first developed a hinge (say) in the rafters, and not until the last hinge was developed somewhere else would it fail, it was possible that we might get away with a more economical foundation. But that was his doubt.

Mr. P. L. Capper (Member of Council), referring to the problem of the foundations said we must consider that there were two stages ; one was the strength of the foundation itself to resist the moments induced by the pressure applied to it and the other was the bearing pressure on the soil. No information was given in the paper as to the condition of the soil or details of the design. But with regard to the bearing pressure on the soil there were again two things to be taken into consideration, i.e., the bearing pressure as regards shear failure and the pressure which would limit the settlement to a tolerable value, which latter was the more difficult to determine. The chief point was that the load factor of the foundation against those types of failure could not be defined with anything like the same precision as the load factor of a steel section subjected to a moment, where the yield stress of the steel was known. A larger load factor should therefore be used in designing the foundation. Again, the distribution of pressure under a foundation subjected to eccentric loading was still a matter for conjecture, and probably it was a wise precaution to design the footing to resist the full plastic moment.

Mr. John King (Associate-Member) said Mr. Callard represented a commercial organisation, and had stated in the paper that he was concerned to put up an economical structure. We knew, of course, that there was a shortage of steel at the time this structure was built, and that this shortage had perhaps tended to create a false factor. Would the speaker be prepared to offer the opinion, that the plastic method of design did offer the client a cheaper structure in the end, in terms of the complete structure, that is including the additional costs involved for special foundations ?

Mr. Callard said that whether one designed a structure elastically or plastically the costs to be

compared must be confined to the steelwork and column foundations. All he could say was that, given similar circumstances, he would be quite prepared to design and construct another building of that type by the plastic method with reasonable confidence that he could do it more cheaply than by the elastic method. In the case of the building described in the paper the price quoted for fabrication and erection of the steelwork designed by the plastic method was less than would otherwise have been paid.

Mr. R. D. TEAGUE (Associate-Member) said he did not quite understand why the bolts were lowered into the foundations with the columns. Usually in the erection of such a structure it was practical to set the bolts in the concrete and to drop the columns on them. It seemed to him that in this case one could only rely on the adhesion of the concrete when the bolts were grouted in to resist the bending moment.

Secondly, the building was surrounded by a 9 in. thick brick wall. He asked if, when designing the foundations, advantage was taken of adding the weight of the wall on the foundations as an offset to the large bending moment. It was customary to put a beam under the wall and thereby transfer the deadweight to the foundations.

Thirdly, he had noticed that 5 per cent. of the roof area was in "starlighting," and he asked why 10 per cent. was not used, for thereby another 100 per cent. more daylight would be gained.

In conclusion, he said the lecture had been really interesting.

Mr. CALLARD replied that indented foundation bolts were used, it being considered at the time that that was an effective way of doing the job. He had known that he would get into trouble if he mentioned the foundations. The walls were carried on a footing which was in turn partly carried on the stanchion foundations and thereby assisted in resisting the overturning moment.

With regard to the lighting, 5 per cent. of the roof area was used for admitting daylight because that was thought to be sufficient. The inside of the roofing material had a light coloured gloss surface so that there was good reflection. Experience had shown that the natural daylight in the building was quite adequate for the operations carried out therein.

Dr. E. H. BATEMAN (Member of Council) asked whether the author had considered the effect of snow loading on half the span of the roof, because it was that condition which occasionally caused the collapse of a building of that type, which was designed for minimum weight and had very slender members. The effect of snow on an arch could be extremely damaging if it was not uniformly distributed, and he suggested that in the case of a structure such as that described the possibility of having snow on half the roof should not be overlooked. It was very much more damaging to that type of structure than to the old-fashioned type of framed roof truss.

Mr. CALLARD replied that that condition had not been considered.

Mr. R. J. WILKINS (Member) asked if Mr. Callard could add to the value of the paper by giving some information on deflection. The plastic theory was designed on the basis of strength, but we needed to know the deflection and he wondered whether any measurements of deflection had been made. The point in using the plastic method was to effect saving in weight and he had been rather surprised by the figure of 34½ per cent. saving and had thought it was nearer 15 or 20 per cent. but that had now been cleared up by earlier speakers in the discussion.

Mr. CALLARD said the actual deflection was not measured and he did not think it could be done now because he was not sure that he had a satisfactory reference point. Dr. Roderick, who was then one of Professor Baker's assistants at Cambridge University had calculated that the deflection under dead plus snow loads would be approximately ¾-inch at the eaves and 9/10-inch at the apex. That was quite a considerable deflection and, of course, deflections would be greater in a portal with pinned bases.

Mr. S. VAUGHAN, (Vice-President) said the discussion had developed a good deal on the question of foundations ; and in view of what had been said it would be extremely helpful if, perhaps in a written reply, Mr. Collard could give the actual size of the foundation that was used and the position of the stanchion on that foundation.

Mr. L. E. WARD (Associate-Member) said that, bearing in mind that the building was designed as a commercial proposition, he felt it would have been much more economical had a haunched type been used. Judging by the photograph of the building in use, such a haunched type would not have been inconvenient, and it would have reduced considerably the deflection, both laterally and vertically.

He also wondered whether the author had considered a deepening of the frame and the use of site bolting instead of site welding, for by that means he believed the welding could have been carried out in the shops giving more control, and erection time could have been reduced considerably. The time from the initial design to the completion of the building was a little over two years, and he believed that that time could have been reduced considerably had site welding not been used.

Mr. CALLARD said that what Mr. Ward had put forward about haunching was probably true. Some comments on that matter had been made by Dr Heyman, that it would have been possible to effect greater economy in the use of steel if they had been energetic and clever enough at the time. Perhaps if they were doing the job again they would take advantage of that.

About deepening the section and then using bolts shop welding on a rib, he said that such a design was not considered. They had accepted a uniform section frame as giving an economical design. Very studiously and carefully they had avoided drilling any holes in the frame whatsoever. It was an interesting question whether one could be bold enough in a plastically designed frame of this type to bolt the sections together choosing the position of the bolted joints in such a way that they corresponded with points of small bending moment, and so get away with it.

Dr. HEYMAN suggested that it tended to be a little uneconomical unless one were careful where one placed the site joints. He knew of two buildings designed in that way. If the joint at the apex or at the eaves were bolted one had to haunch the section considerably and still use fairly large bolts to achieve the strength. If one wanted to go to a haunched design the best way was to cut straight through the haunch and have two bolted division plates. That was rather expensive to manufacture, and there was also the transport problem, since one might have to transport crooked members.

Mr. H. KNOTT, as a representative of the contractors who had constructed the building, amplified one or two of Mr. Callard's answers to questioners.

On the point that the job had occupied two years, he said structural engineers would know what the steel supply poisition was at that time. He added that the time from the starting of the site work and the occupation of the building was about eight months ; the licensing occupied the rest of the time !

The question had been put to Mr. Callard, as to whether he would design another such building as that described in the paper. Perhaps if he had known where the one described would lead him he would not have designed that one ! Perhaps the question could be put more fairly to the contractors, who had tendered for the job in competition. Very rarely had they the opportunity to provide a design, but the occasion did arise and his company were in fact doing another. So that the answer to the question was that the contractors at any rate were prepared to do another.

In regard to the jigging and the method of erection, he did not claim that his company adopted the perfect method. The site was rather enclosed and rather rough. It was workable, but the contractors could not very reasonably lay out the frames complete, weld them and pick them up all in one. They had wanted fairly large jigs and had adopted the expedient of making small jigs for the joints and using for the 60 ft. jigs the columns already placed in position. The columns gave the spacing, and the jigs which were used to locate the rafters on the columns gave the perfect set-up for the apex. That was one of the reasons for adopting that method instead of laying the whole frame out, welding it and lifting it in one.

Mr. CALLARD was very grateful to Mr. Knott for having picked up the points he had mentioned. Although it was stated in the paper that the job began in January, 1950, and was finished in 1952, the construction did not by any means occupy that length of time. Quite a lot of time was spent on design and on getting various permissions, including the building licence.

Mr. J. A. G. SMITH (Member) asked whether Mr. Callard had any ideas on the upper limit of size of cross-section to which the plastic theory should be applied. The cross-section in the present case was 10 in. deep, being a 10 in. $\times$ $4\frac{1}{2}$ in. joist. Would Mr. Callard be prepared to apply the same method to a girder, say, 10 ft. deep ? He felt that the size of cross-section might have some bearing on the applicability of the plastic theory.

Mr. CALLARD said he had not considered the problem of an upper limit in size.

Dr. HEYMAN said there was a plate girder design which had been completed and the fabrication of which was starting. The size was 6 ft.

Mr. CALLARD asked whether there was any theoretical limit.

Dr. HEYMAN did not think so.

Mr. G. A. JONES said that in a building designed according to the plastic theory the stresses under working load were likely to be higher than on one designed according to the normal elastic theory. Therefore it would be necessary to consider, in the case of some buildings at any rate, the possibility of lateral instability of the frame which was subject mainly to bending, as was the building described in the paper. He believed it had been suggested in the past by Professor Baker and his associates at Cambridge that one should work on the present limitation of slenderness ratio of (say) 100. He asked if Mr. Callard or Dr. Heyman could give any help on that point.

Dr. HEYMAN said that the stanchion problem was still under investigation at Cambridge ; it looked as though that work might be nearing completion, and he agreed that some working rules were necessary.

Written Reply by Mr. Callard

In view of the time devoted in discussion to foundations, in answer to Mr. Vaughan's request the attached sketch shows the column foundations as constructed, the column eccentricity being 1 ft. 8 in. The axial load of six tons was the vertical load calculated by summing the weights of the roofing material, frame, snow load, brick buttresses and the weight of the foundation itself. This figure excludes any proportion of the weight of the walls and wall footings transferred to the column foundation.

Book Review

Corrosion—A Series of Papers Reprinted from "Research," Vol. 5, 1952. (London : Butterworth's Scientific Publications, 1953.) 60 pp., $9\frac{3}{4}$ in. $\times$ $7\frac{1}{4}$ in. 6s.

This publication consists of eight papers each written by an authority on the particular aspect of corrosion considered. Some of the papers, while important and interesting, are of no particular value to the structural engineer. This applies to Nos. 2 and 3, dealing with the prevention of corrosion of ships in sea-water, and a detailed explanation of cathodic protection by impressed current. Whilst paper No. 8 has no immediate value, dealing as it does with " Vapour phase inhibitors," their use at present being limited to the field of equipment, tools and instruments, the subject is one which cannot be entirely neglected as its future application may be very wide and far-reaching.

The first paper gives a general survey of the nature of metallic corrosion, immersed, underground and atmospheric, with a brief description of controlling factors and inhibitors in general.

Papers 4, 5 and 6 are undoubtedly the most important to the structural engineer. Paper 4 deals with metallic coatings such as electro deposition, hot dipping, spraying and mechanical processes, whilst 5 and 6 deal with the protective action of all types of paints and inhibitors considered both from the laboratory and also the practical point of view.

The whole of the papers are written very lucidly, the illustrations are particularly clear and the references at the end of each paper being extremely comprehensive and useful.

The whole subject is shown to be one of complexity, and whilst the papers point out that no inhibitor, paint or coating is ideal under all conditions, they do indicate the great progress which has been made in the subject of corrosion and the direction in which future development will have to take place.

R. W. S.

Correspondence

The Institution, whilst being at all times pleased to open its columns to correspondence, cannot accept responsibility for the opinions expressed.

To the Editor of THE STRUCTURAL ENGINEER

A Derivation of Maximum Stanchion Moments in Multi-Storey Frames by Means of Nomograms
R. H. WOOD, Ph.D., B.Sc., A.M.I.Struct.E.*

Sir,—We welcome this paper by Dr. Wood because for the first time in an official publication the non-proportionality of the stanchion moments with the load, for increase of loads above the design loads, is recognised and the irrational basis of the theory published by the Steel Structures Research Committee is clearly demonstrated.

The present method of design envisaged by Dr. Wood is to calculate the bending moments in a rigid frame under the factored load, and then to see that the yield stress is not exceeded on the stanchion.

It seems a pity, however, that Dr. Wood has preferred to set out his calculations in an older mathematical form rather than to use the ideas of modified stiffness (s) and carry-over factor (c), and the more familiar methods of moment distribution. Tables of s and c have been published for a complete range of axial loads and for any particular axial load the stiffness of a stanchion compared with its stiffness at zero axial load and the appropriate carry-over factor can be read off. These are easier to manipulate than the functions X and Y used in the paper, which have the effect of making the calculation for even the very restricted case of Fig. 2 seem difficult when it can in fact be performed very quickly by simple arithmetic (see Appendix).

However, it is in the engineering aspect of the problem rather than the calculations themselves that we believe

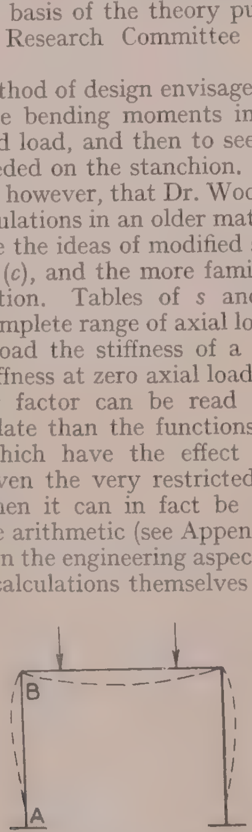


Fig. 11
No sway permitted

some clarification is needed. Dr. Wood does not define his terms "pure double curvature" and "pure single curvature" explicitly but Figs. 3 and 9 indicate that by these terms he means equal rotations of the two ends of a stanchion in like and unlike directions respectively. If this is so, the statement

> the restraining moments must always be in a state of 'pure' Double Curvature at the Euler Load, no matter how unsymmetrical the frame and loading may be. In other words a state of 'pure' Double Curvature is the only possible way of getting past $\alpha L = \pi$ as far as the stanchion length is concerned."

is not true. For example, the frame in Fig. 11 is stable at and above the Euler Load, but A does not rotate and B does; and in Fig. 12 the joints A and B rotate different amounts at the Euler load.

If by "pure" Double Curvature is meant that equal and like moments appear at the end of the stanchion, then still no inference can be drawn regarding sing curvature or double curvature except that at exactl the Euler Load any symmetrical single curvature term will be zero since the carry-over factor is unity, an

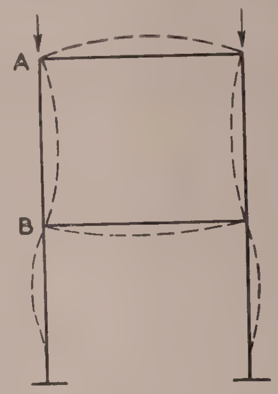


Fig. 12
No sway permitted

therefore all that remains is the unsymmetrical com ponent which because the carry-over factor is unit gives equal moments at the two ends.

At the Euler Load carry-over factor = 1.

SYMMETRICAL (SINGLE CURVATURE) TERMS

		A	B
Applied bending moment	...	$+M$	$-M$
Carry-over	...	$-M$	$+M$
Final bending moment	...	0	0

ANY OTHER TERMS

		A	B
Applied bending moment	...	M_A	M_B
Carry-over	...	M_B	M_A
Final bending moment		M_A+M_B	M_A+M

This is only true at exactly the Euler Load, above th load the moments will not in general be equal, but th structure may of course remain stable if the stanchic is framed to stiff beams, up to two or three times th Euler Load (as Table I, p. 324 shows).

There is another consideration of paramount in portance; choice of a particular load case in no wa determines the elastic instability mode of a structu which depends only on the stiffness of the membe and the axial loads in the stanchions. Hence "pure double curvature has no stabilising effect. It has a apparent stabilising effect in the calculation, but onl if the form of calculation is excluding the real mode failure. Thus if the structure of Fig. 9b becom unstable at a certain load w, then the structure Fig. 9a becomes unstable at exactly the same loa unless an additional fixing force is used to preve lateral displacement of X, the mid height of the sta chion. Obviously this is not provided in the re structure and it fails. (If a check is made in the calc lation X will be found to be unstable against later displacement just as in Fig. 9b; however, checking wi a double curvature case specifically excludes conside tion of lateral instability of X). If it is required t obtain the elastic instability load of a structure a mu

*THE STRUCTURAL ENGINEER, *Vol. XXXI, No.* 11, *Nov.* 1953.

safer way is to test with one moment only and not with either "pure" single curvature or "pure" double curvature.

This does not mean that the type of loading has no effect on the *failure* load of the structure. It has a considerable effect, and Fig. 9b has a lower *failure* load than Fig. 9a because failure depends on both the elastic instability load and the load at which enough plastic hinges form to cause collapse. However, consideration of failure conditions are outside the scope of the paper.

Another matter bound up with the choice of loading case and the critical mode of the structure is the reservation made in the paper that no sway is allowed. In all frames formed of only vertical and horizontal members and in which the cladding does not stiffen the vertical panels, the mode of elastic instability is invariably a sway case which occurs at a value of axial load in the stanchions almost always less than the Euler Load. This means that it would be unsafe to use the data in the paper unless a check is made that the bracing or panel filling is sufficient to prevent sway.

Appendix

As the axial load in a strut increases the stiffness of its ends against rotation decreases. At zero axial load, i.e., for a beam, the slope deflection equation gives the relationship :—

$$M_{AB} = 4E..k\theta_A$$

when end A is rotated through an angle θ_A. In a strut this relationship becomes

$$M_{AB} = s.E..k\theta_A$$

where s is a constant which depends on the axial load carried. The values of s for ranges of axial load have been tabulated[1,2] and widely used. Thus the stiffness of one end of a stanchion becomes $k \times \frac{s}{4}$ instead of k.

In a uniform beam, if a moment is applied at one end, one half of that moment carries over to the other end. In a strut this carry-over factor is not a half but has a value c which rises from $\frac{1}{2}$ at zero axial load to unity at the Euler Load and infinity at about twice the Euler Load. Values of c have also been tabulated[1,2].

Using these values it is possible to carry out normal moment distribution calculations allowing for the reduced stiffness of the stanchions[2,3]. These calculations are so simple that no nomographic solution seems necessary. As an illustration, the example given by Dr. Wood at the bottom of p. 319 is worked below.

The notation used for degree of fixity $\overline{B}$ in the paper is not the best one and to simplify it and bring it in line with normal moment distribution the following notation will be used :

$$\text{Dist. Coeff. } AB = \frac{\text{column stiffness}}{\text{total stiffness at joint}} = \frac{k \times \frac{s}{4}}{\Sigma A}$$

where k is the normal $\frac{I}{l}$ value for the stanchion AB considered.

$\frac{s}{4}$ expresses the reduction in stiffness due to the axial load and ΣA is the total stiffness of all members meeting at the joint A considered (if any other members carry axial loads then their $k \times \frac{s}{4}$ values are used instead of merely k values).

Here $\overline{B} = 0.3$ $\overline{B}' = 0.2$ $\alpha L = 2$ and from tables $\frac{s}{4} = 0.86$ and $c = 0.63$

$\dfrac{k \times \frac{s}{4}}{\Sigma A}$ if calculated directly would have been

$$= \frac{.3 \times .86}{.7 + .3 \times .86} = .27$$

$$\frac{k \times \frac{s}{4}}{\Sigma B} \qquad \frac{.2 \times .86}{.8 + .2 \times .86} = .18$$

The example now follows :—

C.O. = .63

	A			B
Distribution Coeff.		.27	.18	
1. Balance unit moment at A	—.73	—.27		
2. Carry-over to B			—.17	
3. Balance B			+.03	+.14
4. Carry-over to A		+.02		
5. Final Moments	—.73	—.25	—.14	
	—.98			
6. Moment at A brought back to unity	—.75	—.25	—.14	
7. Balance unit moment at B			—.18	—.82
8. Carry-over to A		—.11		
9. Balance A	+.08	+.03		
10. Carry-over to B			+.02	
11. Final Moments		—.08	—.16	—.82
			—.98	
12. Moment at B brought back to unity		—.08	—.16	—.84

For unit equal out of balance moments

$$M_A = -.25 - .08 = -.33$$
$$\& \quad M_B = -.14 - .16 = -.30$$

For any other out of balance moments the result would have been obtained by scaling up lines 6 and 12 in the correct proportions.

This method avoids the need to interpolate between values obtained from two nomograms and the reduced stiffnesses of adjacent stanchions are automatically allowed for in the original distribution coefficients. There is, as of course there should be, no indeterminacy at the Euler Load. This arithmetical method is also more readily extended to practical structures.

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[2]W. Merchant, "The failure load of rigid jointed frames as influenced by stability." Journal Inst. Structural Engineers, July, 1954.

[3]N. S. Hoff, B. A. Boley, S. V. Nardo, and S. Kaufman, "Buckling of rigid jointed plane trusses." Proc. ASCE, Vol. 76, June, 1950.

Yours, etc.,

W. MERCHANT (Associate-Member).
A. BOLTON (Graduate).
College of Technology,
Manchester.

DEC. 1953

Institution Notices and Proceedings

HONOURS AND AWARDS

In offering their sincere congratulations to the following members on the distinctions recently conferred upon them, the Council feel they are also expressing the good wishes of the Institution :—

KNIGHT BACHELOR

Major A. H. S. Waters, V.C., C.B.E., D.S.O., M.C. (Past President).

ORDER OF THE BRITISH EMPIRE—O.B.E.

Mr. F. S. Snow (Past President).

PRESIDENTIAL ADDRESS

The Presidential Address for the Session 1954-55 will be given by Dr. S. B. Hamilton, A.R.C.S., M.I.C.E., M.I.Struct.E., on Thursday, October 7th, 1954, at 6 p.m.

EXAMINATIONS—JANUARY, 1955

The Examinations of the Institution will next be held at centres in the United Kingdom and Overseas on January 11th and 12th, 1955 (Graduateship), and January 13th and 14th (Associate-Membership).

REPRESENTATION

The Council have appointed the following Institution Representatives :

ROYAL INSTITUTION OF CHARTERED SURVEYORS : CODE COMMITTEE ON ZINC COVERINGS
Mr. C. E. Cannons (Member).

CODE OF PRACTICE COMMITTEE FOR CIVIL ENGINEERING :
Mr. Walter C. Andrews, O.B.E. (Past President).
Mr. Ernest Granter (Past President).

CODE OF PRACTICE COMMITTEE FOR BUILDINGS :
Mr. T. Bedford (Member).
Mr. L. E. Kent (Hon. Treasurer).

WEST MIDLANDS ADVISORY COUNCIL FOR TECHNICAL EDUCATION—CIVIL ENGINEERING ADVISORY COMMITTEE :
Mr. G. S. McDonald (Vice-President).

BIRMINGHAM EDUCATION DEPARTMENT—BUILDING PROFESSIONAL COURSES ADVISORY COMMITTEE :
Mr. E. R. Deeley (Associate-Member).

The Council have also nominated the following member to serve on B.S.I. TECHNICAL COMMITTEE—STRUCTURAL SECTIONS FOR LIGHT ALUMINIUM ALLOYS :—
Professor A. G. Pugsley, O.B.E. (Vice-President).

RESEARCH AWARDS

The Council has instituted a Research Prize Fund, from which awards may be made annually to the author or joint authors of papers describing original research which they have carried out. Research awards may be made for papers read at Headquarters or in the Branches and published in the Journal, or for papers published in the Journal only without being read at an open meeting.

The assessment for such awards will be made annually, but awards will be made only to the contributors of such papers as reach a standard judged by the Literature Committee to be satisfactory.

Work submitted under this scheme must be original and may include any of the following :—

(a) investigations of an experimental or analytical character ;

(b) studies of historical or statistical records ;

(c) improvements in principles or methods of construction ;

(d) research into methods of structural engineering and building, the nature and use of plant and the organisation of engineering work ;

(e) any related or combined studies which are deemed by the Literature Committee to be of a research character.

In cases where the research work described in the paper was not the work of one individual, the names of all the collaborators should be given in the paper.

Awards may take any or all of the following forms :—
A research medal ; a diploma ; a money prize.

Application for consideration for a research award must be made to the Secretary of the Institution, and in preparing papers for reproduction in the Journal, authors must comply with the conditions laid down for all such contributions. Particulars of these conditions may be obtained from the Secretary.

In judging research papers, the following factors will be considered :—

(a) the nature of the subject and its conclusions ;

(b) the value of the paper in advancing the science and art of structural engineering ;

(c) the standard of preparation and orderly arrangement of the subject-matter.

Research papers will also be eligible for adjudication for the Institution Medals if they comply with the Regulations governing those awards.

The closing date for the receipt of applications in respect of papers published in the Journal between October, 1953, and September, 1954, is October 1st, 1954.

MACLACHLAN LECTURE COMPETITION, 1955

The closing date for the receipt of entries for the next Maclachlan Lecture Competition is Thursday, March 31st, 1955. The general conditions of the competition are as follows :

1. The Institution of Structural Engineers shall institute a written lecture to be known as the Maclachlan Lecture and to be held annually.

2. The subject of the Lecture may be on any aspect of Structural Engineering as long as in every second year the subject shall be confined to steel structures. (This will be the case in 1955.)

3. Entrance into the competition for the Lecture shall be confined to Associate Members of the Institution, who are under the age of 32 years.

4. All papers entered for the competition shall be submitted to assessors to be appointed by the Council of

the Institution, and all such papers (including the prize-winning Lecture) shall be available for publication in the Journal of the Institution at the discretion of the Council.

5. No paper submitted shall have been published or read elsewhere.

6. The winner of the competition shall be required to present the Lecture to a meeting of the Institution at which he will be presented with the sum of £17 10s.

7. Should a competitor's paper be considered worthy of ranking second in merit he will receive a consolation award of £5.

8. In the event of there being no winner of the competition in any one or more years, whether because no lecture submitted is considered to be of sufficient merit to warrant award, or for any other reason, the Institution shall transfer the above sums to the Research Fund of the Institution.

PARTICULARS OF THE COMPETITION FOR 1955

1. The Maclachlan Lecture will be given at a meeting of the Institution to be arranged towards the end of 1955.

2. The subject of the Lecture shall be confined to steel structures.

3. The work should be submitted as the script of a lecture which the author, if successful in the competition, will deliver before an audience in the course of about one hour. The development of mathematical formulæ and detailed calculations should be avoided as far as possible in the text ; if they are essential they should be embodied in appendices. Photographs, drawings, graphs, etc., which would appear as illustrations to the lecture in published form, should accompany the script. If additional illustrations would be shown as slides, a list of these should be included.

4. Six copies of each Lecture should be submitted and should be addressed to the Secretary of the Institution.

5. The closing date for the receipt of entries by the Institution is Thursday, March 31st, 1955.

LONDON GRADUATES' AND STUDENTS' SECTION

A visit to London Airport has been arranged for the morning of Saturday, August 21st. A coach will leave the Institution at 8.45 a.m. The return fare is 7s. 0d. The number of visitors is limited to 35, and those wishing to participate should make application, enclosing the coach fare, to the Hon. Secretary, J. A. Pope, 53, Cranleigh Drive, Leigh-on-Sea, Essex.

A visit to the Atomic Research Establishment at Harwell has been arranged for Saturday, September 18th. The number of visitors is limited to 24, of British nationality. Transport by coach from the Institution has been arranged and details will be sent to those who wish to participate in the visit, on application to the Honorary Secretary.

The next indoor meeting of the Section will be held at 11, Upper Belgrave Street, London, S.W.1, on Tuesday, October 19th, at 6 p.m., when Mr. L. Scott White, O.B.E., M.I.C.E. (Past President) will read a paper on "Government Offices, Whitehall Gardens ; the special problem of the re-siting of an historic building." This will be followed by a visit on Saturday, October 23rd, at 10 a.m., to the site of the work to inspect the historic building, together with new construction now in progress.

BRANCH NOTICES

LANCASHIRE AND CHESHIRE BRANCH

A party of 15 members of the Branch visited the steelworks of Messrs. John Summers & Sons, Ltd., at Shotton, on June 14th. The party was conducted over the coal stocking plant, coke oven battery, and by-products plant, where construction on the extension was still in progress. The blast furnace was then visited and also the melting shops, rolling mills and hot and cold strip-mills.

It was a most interesting and enjoyable visit and members of the Branch are grateful to Messrs. John Summers & Sons, Ltd., for permitting it and for their courtesy in conducting the party over the works.

Joint Hon. Secretaries : A. S. Sinclair, A.M.I.Struct.E., 17, The Circuit, Cheadle Hulme, Cheshire ; M. D. Woods, A.M.I.Struct.E., 58, Spring Gardens, Salford, 6.

MIDLAND COUNTIES BRANCH

Hon. Secretary : L. A. Firminger, A.M.I.Struct.E., 656, Chester Road, Erdington, Birmingham, 23.

GRADUATES' AND STUDENTS' SECTION

Hon. Secretary : H. L. Bramwell, 139, Wood Lane, Handsworth Wood, Birmingham, 20.

NORTHERN COUNTIES BRANCH

Hon. Secretary : O. Lithgow, A.M.I.Struct.E., 4, Stoneleigh Avenue, Acklam, Middlesbrough.

NORTHERN IRELAND BRANCH

The following Honorary Officers and Committee members have been elected for the Session 1954-1955 :—

Chairman : Mr. J. C. Malcomson, B.Sc.

Vice-Chairman : Mr. S. G. Duckworth, J.P.

Immediate Past-Chairman : Mr. M. C. Gillies.

Hon. Auditors : Messrs. R. Montgomery and S. O. Morton, B.Sc., A.M.I.C.E.

Hon. Secretary : Mr. A. H. K. Roberts, B.A., B.A.I., M.I.C.E.I., "Barbizon," 26, Dunlambert Park, Belfast.

Hon. Assistant Secretary : Mr. J. McClure, B.Sc., A.M.I.C.E.

Committee : Messrs. R. Ferguson, B.A., B.E., M.I.C.E., W. A. Plester, T. A. N. Prescott, M.Sc., A.M.I.C.E., R. J. N. Sweetnam, M.A., M.A.I., A.M.I.C.E., J. M. C. Tyack.

SCOTTISH BRANCH

Hon. Secretary : G. Drysdale, A.M.I.Struct.E., "Niaroo," 33, Union Street, Motherwell, Lanarkshire.

SOUTH-WESTERN COUNTIES BRANCH

The Annual General Meeting of the South-Western Counties Branch, was held at The Duke of Cornwall Hotel, Plymouth, on May 21st, 1954.

The following Hon. Officers and Committee members were elected for the Session 1954-55 :—

Chairman : Col. R. Hazzledine, O.B.E., T.D.

Vice-Chairman : Mr. E. W. Howells.

Hon. Auditors : Mr. H. J. Scoles, L.R.I.B.A.

Joint Hon. Secretaries : Mr. E. W. Howells, M.I.Struct.E., 10-12, Market Street, Torquay, Devon; Mr. C. J. Woodrow, "Elstow", Hartley Park Villas, Tavistock Road, Plymouth.

Committee : Lt.-Col. F. J. Dean, Messrs. H. W. G. Miller, F. W. Potter, F. J. Powell, M.B.E., L.R.I.B.A., F.R.I.C.S., H. Toft, W. C. Tyler, F. M. Upson, L. F. Vanstone, L.R.I.B.A.

The Branch Programme for the next Session was discussed at length and various suggestions were adopted.

WALES AND MONMOUTHSHIRE BRANCH

Hon. Secretary : K. J. Stewart, A.M.I.C.E., 15, Glanmor Road, Swansea, Glam.

WESTERN COUNTIES BRANCH

Hon. Secretary : E. Hughes, 23, Southdown Road, Westbury-on-Trym, Bristol, 9.

YORKSHIRE BRANCH

Hon. Secretary : E. Wrigley, A.M.I.Struct.E., 17, The Drive, Alwoodley, Leeds.

UNION OF SOUTH AFRICA BRANCH

Hon. Secretary : A. E. Tait, B.Sc., A.M.I.C.E., P.O. Box 3306, Johannesburg, South Africa.

Natal Section Hon. Secretary : E. G. Bennett, A.M.I.Struct.E., c/o The Reinforcing Steel Co., Ltd., P.O. Box 49, Merebank, Durban.

Cape Section Hon. Secretary : R. Stubbs, M.I.Struct.E., The Reinforcing Steel Co., Ltd., P.O. Box 2692, Cape Town.

ADDITIONS TO THE LIBRARY

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Autogenous Healing of Concrete in Compression

By E. F. Whitlam, M.Sc.(Eng.), A.M.I.C.E., A.M.I.Struct.E.

Summary

Concrete that has failed in compression (or tension) possesses the property of healing, providing the fractured parts are maintained damp and in intimate contact. The experimental work described in this paper was carried out to ascertain if any quantititive results could be found concerning healing in compression and how it was connected with the general hardening process in concrete. Only one mix of concrete was used and initial tests were made to ascertain the most suitable degree of failure for specimens to be healed. Concrete cylinders 5 in. dia. × 10 in. high were used. A series of these was tested at varying ages and re-tested after further periods of curing. Load-compression (or deformation) curves were taken for each test. It was found that the healing followed the same form as the general hardening process in concrete and it is thought probable that the healing is dependent upon the damage sustained by the initial compression test. A possible law for the deformation of concrete up to the point of failure is suggested for ordinary and healed concrete. Finally, practical consideration is given to the healing process in everyday site work.

Introduction

The fact that concrete will heal after failure in compression has been known for about 30 years. Attention

Fig. 1.—Mould for concrete cylinders

was first called to the phenomena in 1913[1, 2]. In an article in CONCRETE in 1925[4] the name "Autogenous Healing" was first applied and later there were further reports of healing[5]. Experiments were carried out in America on autogenous healing[6] and later on healing after tensile failure[7]. It has been suggested[9] that "although some engineers do not place much significance on the healing process, if it could be speeded up it would be a great step forward in the study of concrete." There is little recorded information on the subject of healing, and in general it is confined to the observations that healing takes place and that healed specimens are often stronger when re-tested than when first tested.

Scope of Tests

It was decided that a series of cylinders should be cast and tested at ages varying from 3 days upwards to

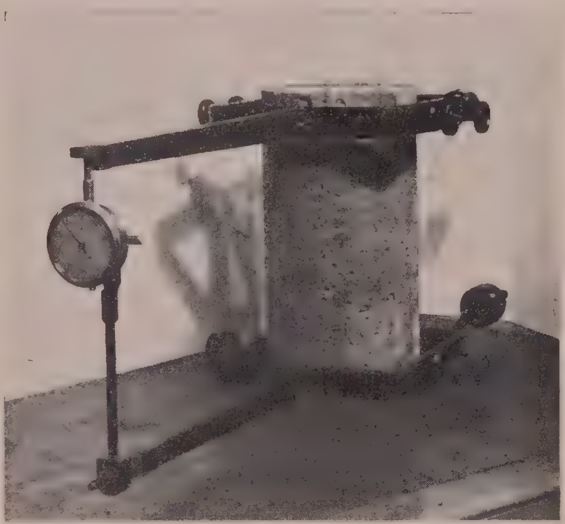

Fig. 2.—Front view of compressometer (Top locking nuts in position)

1 year. The degree of failure was to be determined by preliminary experiments. The strain required was one such that the specimen was undoubtedly in a state of failure but had not completely disintegrated. It could not be said in advance whether this would yield a completely cracked test piece or whether it would merely be indicated by the inability of the specimen to carry further load. Following the tests, specimens were to be cured in water for varying periods of time, generally comparable with the age at the first test, and subsequently re-tested. In actual fact it was possible to give several specimens a further curing and a second re-test, or third compression test.

Moulds for Specimens

One of the moulds is shown in Fig. 1. It was a steel cylinder with a vertical gap in one side, made by a cut approximately 1/32 inch in width. Lugs welded on to the sides of the cylinder on either side of the gap and fitted with ½-in. dia. bolts enabled the mould to be tightened closing the gap. This small springing movement facilitated demoulding when the bolts were released. The baseplate had a small circular upstand over which the cylinder fitted, and against which it was tightened. In the assembled position the internal dimensions were 5 in. dia. × 10 in. height. Four ¼-in. dia. B.S.W. screws were fitted into holes in the mould to provide holes for fixings for the compressometer. The portion of the screw projecting into the mould had been turned down to ⅛-in. dia.to give a hole for a rawl-

plug. These screws were lightly oiled with the mould before a cylinder was cast. The screws can be seen by the baseplate in Fig. 1.

Preparation of Specimens

A concrete mix of 1 : 2 : 4 by weight was employed, using a rapid hardening Portland cement. The coarse

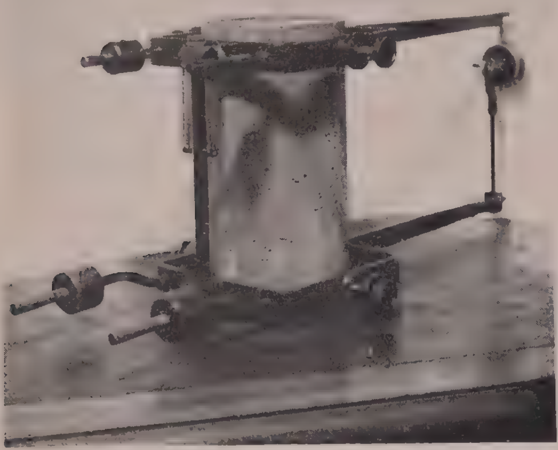

Fig. 3.—Rear view of compressometer

aggregate was a $\frac{3}{8}$-in. max. graded shingle which generally gives a weaker mix than a $\frac{3}{4}$-in. aggregate. The sand contained a fair proportion of fine material. The maximum load capacity of the Riehlé testing machine was 100,000 lb. (about 5,000 lb./sq. in. on a 5-in. dia.

Fig. 4.—Typical specimen after failure

specimen) and it was considered that the leaner mix resulting from the us[illegible] aggregate would be advantageous, as concret[illegible] ths at 1 year would then be of the order of 4,000-5,[illegible] lb./sq. in. with a water cement ratio of 0.6 and 3,000-4,0[illegible] lb./sq. in. with a ratio of 0.7 and allowing for the fact that cylinders and not cubes were being used[10]. This rather high water content was suitable for the aggregates owing to the amount of fine material. All the aggregates were oven-dried before mixing, and appeared to be in agreement with the D.S.I.R. Road Note No. 4, bearing in mind the use of $\frac{3}{8}$-in. aggregate.

Generally four specimens were prepared at a time and hand mixing was employed. A small quantity of sand and cement mortar was placed at the top and bottom of each specimen to give an even surface and the concrete was placed in layers of $1\frac{1}{2}$ in. Each layer was punned with 20 strokes of a 1-in. dia. rod weighing about 4 lb., and 20 blows were applied around the periphery of the steel mould for each layer. It was found that this method of consolidation was satisfactory. A very slight amount of water seeped out at the base of the mould

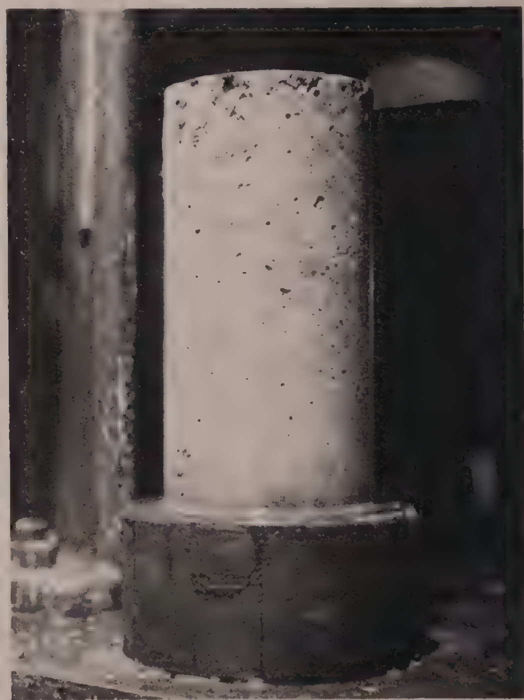

Fig. 5.—Specimen after first compression test

and only a few air bubbles occurred on the face. Care was taken to ensure that the concrete flowed solidly round the steel pins for the rawlplugs. In the case of the cement mortar at the top of the mould a slight excess was used and the top was brought to a level surface by drawing a steel straight edge across the top of the steel mould. A satisfactory result was obtained by doing it four times, twice each in each of two directions at right-angles and using a slight sawing motion with the straight edge. The question of finishing the tops of cylindrical specimens has been the subject of much experimental work. The method of grinding the ends[8], may be suitable but unfortunately it was not possible to try it in this case. Several methods were tried and tested before using the one described. All specimens were demoulded at 24 hours and cured in a water tank at 60°F. The temperature was found to vary little throughout the year. When required for testing cylinders were taken from the tank and left for a few minutes to allow the surplus water to drain away.

Initial Tests

Preliminary tests were carried out to determine what degree of failure should be used on specimens. Com-

pression curves were plotted as testing was carried out. The result of various tests showed that a strain of 0.2 per cent. or 0.002 usually resulted in failure of the specimen. The actual deformation was 16 thousandths of an inch on the 8-in. gauge length or 80 division

Fig. 6.—Specimen after third compression test (twice healed and retested after first test)

creep was taking place. At the time of these experiments only one comment of the question of ultimate strain had been noted[10].

Measurement of Compressions

The compressometer used was designed by Mr. V. C. Davies, B.Sc.(Eng.), M.I.Mech.E., and is illustrated in Figs. 2 and 3. The main frames were of mild steel, fixed to the specimen by screws with milled heads and

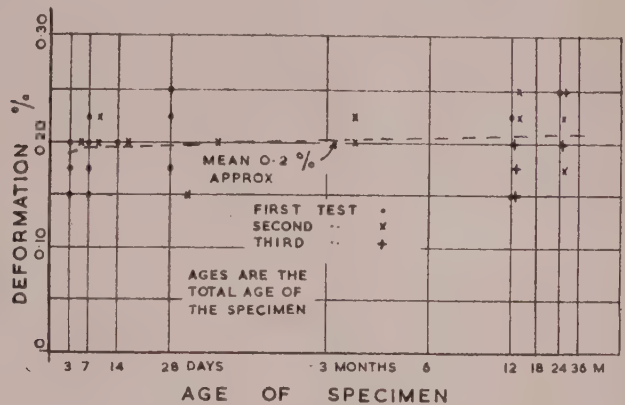

Fig. 8.—Deformation at initial failure or ultimate strain

locking nuts. The motion was balanced by four counterbalance weights seen in Fig. 3. The lower frame carried a vertical ½-in. dia. pillar with a shouldered, rounded top which fitted into a cup seating on the upper frame, being maintained in position by tension springs. This was the member of fixed length against which movement was measured. A standard thousandths dial gauge was bolted to the other end of the frame. The lever arm

Fig. 7.—Various specimens after healing, showing deposits of salts on the sides Specimen at extreme right is control specimen

on the dial gauge and it was decided after careful consideration to use this as the limit. All specimens would be taken to this strain as nearly as possible. The rate of increase of load at this strain was small and in general the needle on the dial gauge was moving steadily, and often while a balance was being obtained

of the compressometer gave a ratio of five to one, giving readings to one-fifth of a thousandth of an inch or two ten-thousandths of an inch. The instrument was fixed to the specimen by means of four purpose made screws. Each of these was made from a No. 8 woodscrew brazed to a hexagonal brass nut. The dead centre of the nut

was drilled to give a pop mark and the screws were fixed into rawlplugs in the specimens using a purpose-made tool. The design of the screws enabled them to be used repeatedly without damage to the pop mark. They can be seen in Fig. 6. The instrument was free of blacklash and was extremely sensitive.

The Riehlé machine used for testing was of the parallel plate type operating on two square thread screws,

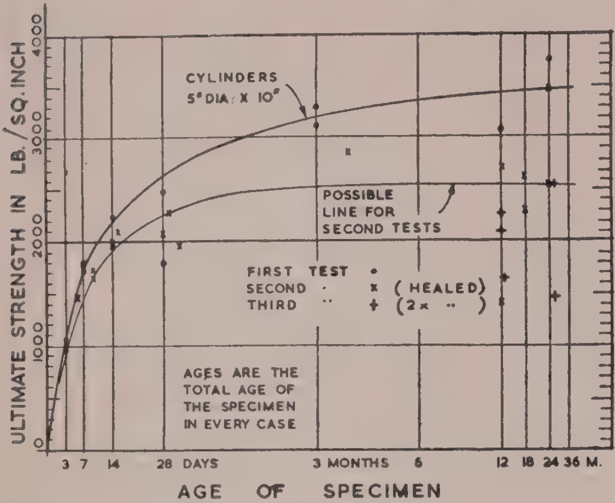

Fig. 9.—Ultimate strength of concrete cylinders

versy over the question of packing, although it is usually employed when the end surface of the concrete is not perfectly smooth. The most satisfactory solution would appear to be that of grinding the ends true[8] when the packing can be omitted. In actual testing of cylinders the dial gauge was adjusted to zero by means of the small screw at the head of the pillar (see Fig. 3). When fixing the special screws to the specimen they were driven to within 1/16 in. of the concrete face, as in early work it was found that when driven flush with the concrete, cracking occurred in a few cases and rendered the specimen useless. It was ascertained that the gauge was reading satisfactorily by tapping both the upper and lower frames to see that the dial gauge returned to the zero position each time. A small load (of the order of 500 lb.) was applied and slow speed loading used. Following this, loading was applied at intervals of about 2,500 lb., and intervals of 5,000 lb. for specimens older than 14 days. Loading was continued until the initial failure point was reached, the increments of loading being reduced in this region. Following failure the load was steadily removed. The screws were removed and the specimen was then placed in the water tank for such time as was thought suitable for the healing.

The same procedure was adopted with re-tests, although in some cases packing had to be used in the rawlplugs as they had hardened with continued immersion in water, and the screws would not take satisfactorily, presumably due to the action of calcium salts on the rawlplug itself. In several second and third

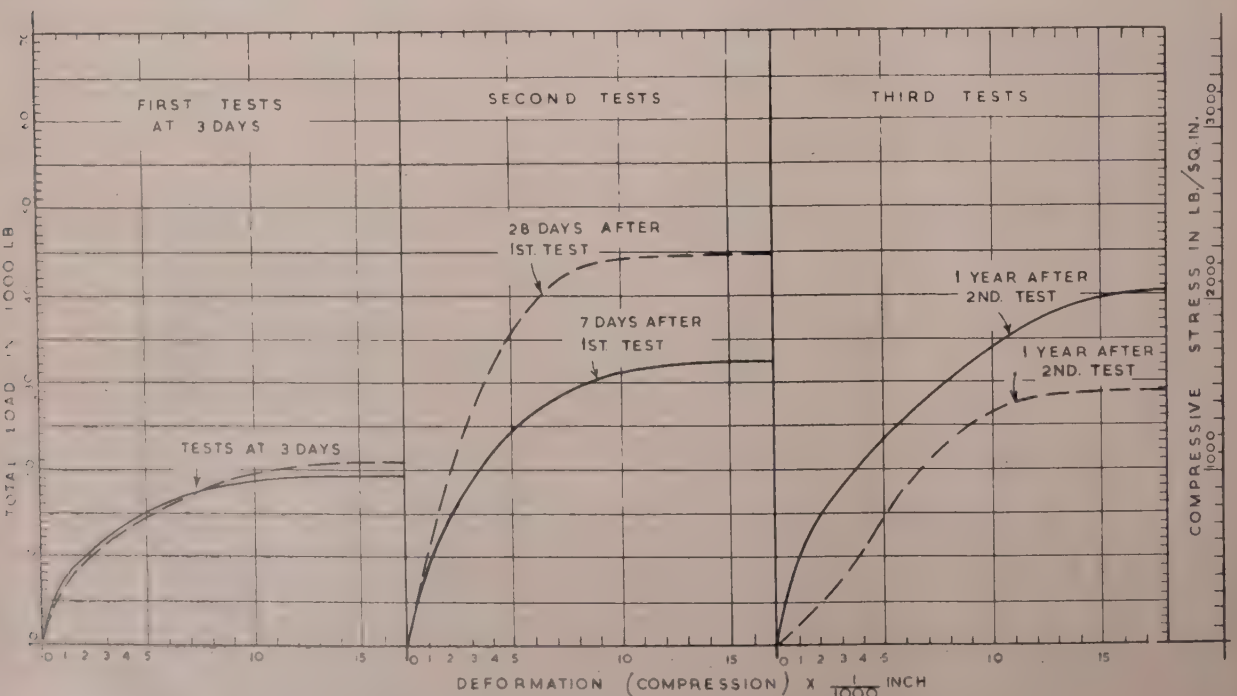

Fig. 10.—Deformation of healed specimens first tested at 3 days

[illegible] by means of the usual type of counterbalance [illegible] mechanical and could be carried [illegible] at different speeds, including a very slow speed [illegible] by a hand clutch. This was used for the [illegible] the smallest unit of loading was 10 lb. Readings were taken in thousands of pounds to two places [illegible] e.g., 10.[illegible] thousand lb. The last figure, however, was generally neglected.

A packing was used at the top of each specimen when testing. A thin piece of woven machine belting was used. There has been (and still is) considerable contro-

tests, however, cracking which had occurred at the top of the specimen prevented the screws being fixed, and further compression readings could not be obtained. It is thought probable that the tops of the specimens were very slightly out of true and this resulted in cracking during tests. When such cracks reached the rawlplug it was not possible to refix the compressometer screws.

Results

In all over 40 specimens were tested but not all of them were suitable for testing a second time and only in a

limited number of cases could third tests be applied. A number of general observations were made :

1. Healing took place in all specimens whatever the age (within the limits of these experiments).

2. The strength at second test was often much greater than at the initial test.

3. At a strain of 0.002 or 0.2 per cent. practically all specimens were in the same state of failure. With very green specimens the value was slightly lower and with some old specimens it was slightly higher (0.0025 or 0.25 per cent.). This state of failure was as previously stated, not that of complete disintegration, although in some cases pieces had spalled off the sides of the specimens (see Figs. 5, 6 and 7).

4. Deposits of white crystals were present on the faces of the specimens in all cases where they had been subjected to any appreciable period of re-curing. These can be seen in Figs. 6 and 7.

5. The deposits appeared at visible cracks but not solely there. They appeared at various other places, probably where minute cracks existed.

6. Most specimens had numerous visible fractures at the top and fairly frequently there were cracks at the base as well.

7. From an inspection of specimens that had been tested three times it looked as though they could be tested again.

8. Compression of specimens under re-test took place in the same manner as initial tests and no peculiarities were noticed.

9. In one case only, a specimen that had been tested was left dry for a period and not in water. On further testing this specimen had practically no strength, which seems to indicate that water is a necessary agent for curing.

The first two of these points bear out previous evidence of healing, although the important point is that a healed specimen can never attain a strength greater than the strength that it would have attained if it had been undamaged. The question of ultimate strain in concrete appears to be one of considerable importance.[11]. The values of ultimate strain are shown in Fig. 8.

Reference has been made[5] to salts deposited on the healed concrete. It is apparent that some chemical action takes place between fractured surfaces that occur within the concrete mass. Whether it is the same action that takes place in the general hardening process in concrete or not cannot be said, but both processes require water to be present. This research was not involved with concrete chemistry, but even so it is presumed that certain of the complex calcium silicates or aluminates which are said to exist in a colloidal form re-form, or re-bond the fractured surfaces together. This appears to be the most probable explanation of the process. The fracturing of the top of specimens is possibly due initially to slight unevenness of the top but some specimens were sufficiently failed to have cracks at the base and were in quite a shattered state, needing careful handling.

Figs. 4, 5 and 6 show three specimens at testing. In Fig. 4 the specimen had been taken to complete failure showing the usual conical form and indicates the concrete texture, while Figs. 5 and 6 show specimens after first test and after third test. Fig. 7 shows the salts deposited on the concrete surface during re-curing. On the extreme right was a control specimen which was not tested but kept for visible comparison. Fig. 9 shows the ultimate strengths of the specimens which were satisfactorily tested. The initial strengths follow the usual form although one or two points are out of line. From the values it is evident that the initial estimate of strengths for this concrete were reasonable as the general strength at 1 year is over 3,000 lb./sq. in. Specimens at re-test do not, however, show a very definite relation-

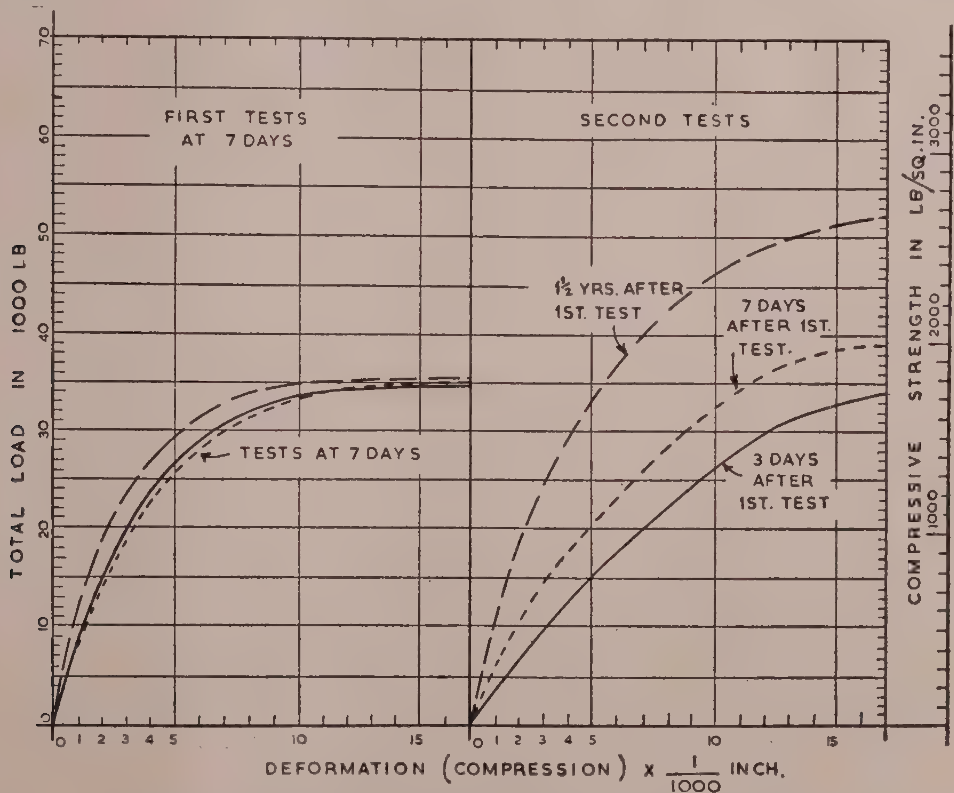

Fig. 11.—Deformation of healed specimens first tested at 7 days

ship. It appears that there may be a mean line at about 80 per cent. of the first strengths. It is considered that the strength of the failured specimen when re-cured is some function of the initial damage sustained at the first test. Unlike tensile tests it is not possible to reach a very definite failure point except that of complete disintegration. The tendency however is for the strengths to increase with age and it is possible that the strength is dependent on the total age of the specimen. Figs. 10, 11 and 12 show compression curves for 3, 7 and 28-day specimens at test and second test with a third test in the cases of 3 and 28 days.

It is noted that the curves all follow the same form although at the third tests there is a tendency for the curve to be rather flatter. These were selected as being reasonably representative of the results. In the second

curves for concrete. Morsch gave the following relation :—

$d = as^r$

where d = unit deformation.
a = a constant
s = unit stress in concrete and
r = an exponential

and the following has been given[3]:—

$s = Kd^n$

where s = unit stress in the concrete
d = unit deformation
K = a constant depending on the strength of the concrete and
n = an exponential, approx. constant.

Most of the experimental work on which these are based takes the compression within working stresses and

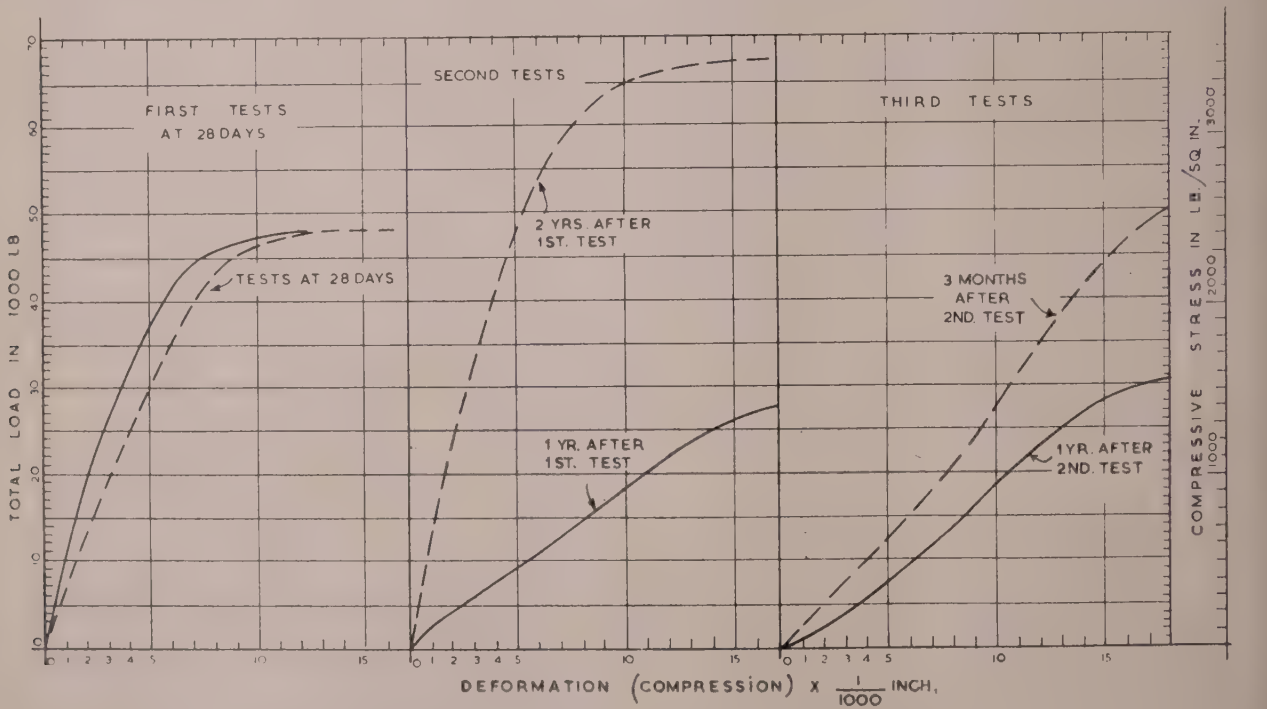

Fig. 12.—Deformation of healed specimens first tested at 28 days

test of the 28-day values there is quite a discrepancy between the values. Presumably the specimen re-tested at 2 years and subsequently at 3 months did not suffer such severe damage at initial failure as the other although after third tests the ultimate strength of 1,600 lb./sq. in. is quite high, when it is remembered that it had already failed at 2,450 lb./sq. in. and 1,400 lb./sq. in., an aggregate total of 5,450 lb./sq. in. The slight dip in the curves at third compression test may be due to a slight slip of the fractured internal concrete surfaces on one another after the healing. This amount would be extremely small. Fig. 13 shows deformation curves for all the specimens tested and the general trend of the curves can be seen. The figures at the right-hand side indicate very approximately the total ages of specimens, this being the total cured age at first, second or third test. It will be noted that the curves all rise steeply at first, flattening out generally after a strain of 0.0005 or 0.05 per cent. Failure takes place at a strain of about 0.002 or 0.2 per cent. ; observation on this value has been made.[11]

Equation of Stress-Deformation Curves

For a long time various formulæ have been offered as representing the relationship for stress-deformation

although the equation can represent the condition it has certain overall limits. Fig. 14 shows the curves of Fig. 13 plotted to log-log scales and the range $d = af^r$ is indicated although it is not well defined. The author considers that there would be advantages in a law which takes the compression right up to the point of failure. In Fig. 15 the curve $y = \tanh x$ is shown. This does not necessarily represent the curves shown in Fig. 13 but the similarity is seen. The reason for the use of the *tanh* curve is that it is one of the exponential forms and it proceeds to a definite limit. It is suggested that a law of the following form may hold.

$$\frac{C}{U} = K \tanh d \text{ or } K \tanh^n d$$

where C = stress in concrete
U = ultimate strength of concrete
d = deformation at stress C
k = a constant and
n = an exponential

The value of n would be dependent on the concrete, generally appearing to be about 1 ; under 1 for richer mixes or a little above for healed concrete and weaker mixes. There is one important point however. The rate of loading affects the form of the compression

considerably[11] and slow or sustained loading produces a much flatter curve. From this it is concluded that K and n would also depend on rate of loading.

The values of n above 1 and possibly up to 2 would produce a curve that reaches the ultimate deformation at the same point generally but starts with the pronounced dip that is seen in the extreme right-hand curves of Figs. 10 and 12. The illustration that such a form is followed presents difficulties, as it is necessary to know the exact ultimate strength of each specimen. The following method was adopted to indicate that several of the curves appeared to follow the relation, and in Fig. 16 the curves previously shown are plotted. Both axes vary logarithmically and the y axis was plotted so that the scale value varies as $\tanh x$, thus :—

$\tanh$ 0.203 = 0.2
,, 0.868 = 0.7
,, 2.994 = 0.99, etc.

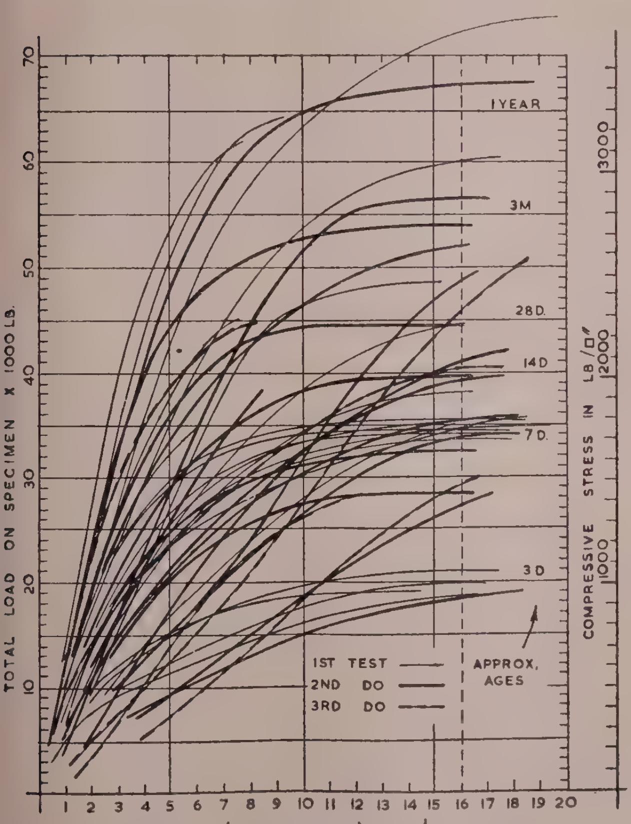

Fig. 13.—Load-deformation curves

and the values 0.203, 0.868, 2.994 plotted on the log scale, were indicated as 0.2, 0.7 and 0.99. When plotted to this scale the curve $y = \tanh x$ becomes a straight line. In Fig. 17 the tests shown as representative samples in Figs. 10, 11 and 12 have been plotted for comparison, and it is seen that they are in keeping with the relation. As the load scale is taken as a fraction of the ultimate of any cylinder it is to be expected that variations may occur in the region of 95 per cent. (and over) of the ultimate load. From such a limited series of experiments it is not possible to draw full conclusions, but it is considered that a law of foregoing form offers advantages to the usual stress-deformation relation or one involving the modulus. The ultimate strength of concrete can be found from either cubes or cylinders, although such results do not necessarily give the exact ultimate strength of the structure which they represent. If suitable values of K and n are known the stress can be found by measuring the deformation and the method is applicable up to the point of failure. Although this is confined to the question of pure compression, it does not exclude the measurement of the compressive stress in members in bending. In such cases known values of K and n would replace the doubtful modulus.

Conclusions

The following conclusions have been reached. Healing takes place in concrete specimens at ages of from 3 days

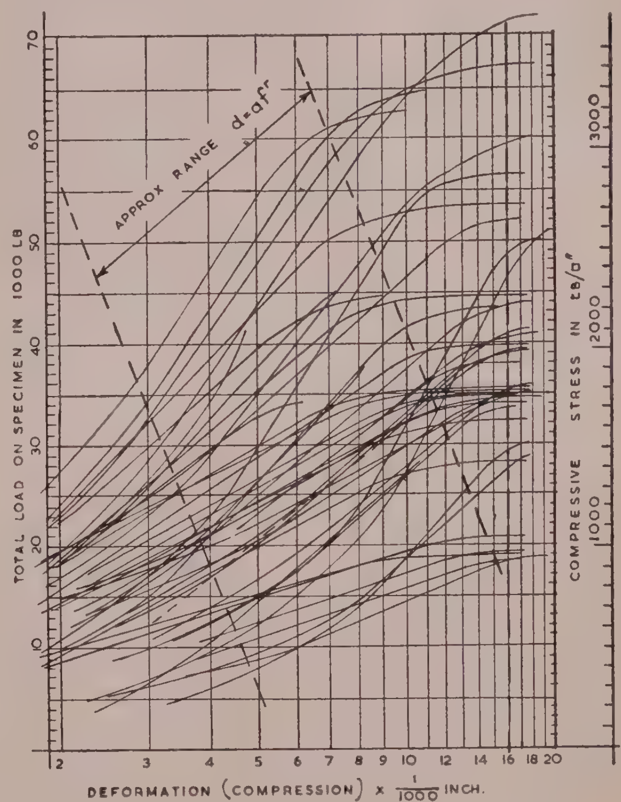

Fig. 14.—Load-deformation curves

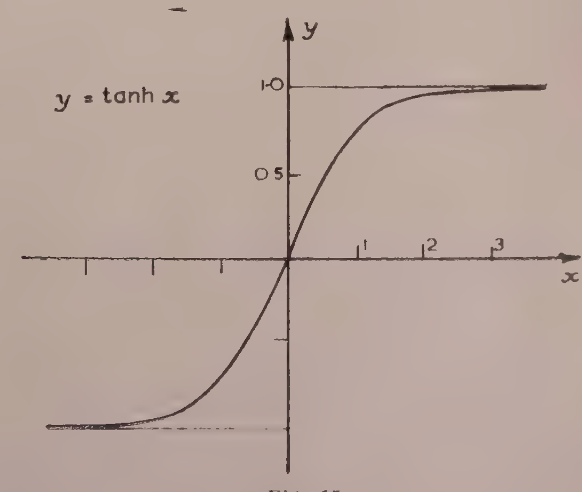

Fig. 15

to 3 years of age. It probably takes place beyond this period but this was the limit of the experimental work. A number of healings can take place and it would appear that the healed strength is dependent on the damage sustained during testing, and is a function of the total age of the specimen, although no particular relationship

appears to exist between healed strength and original strength. The ultimate deformation of concrete in compression is in the region of 0.2 per cent. or 0.002 and this figure holds for most all ages of concrete whether healed or original concrete. The stress in concrete can be expressed in terms of the ultimate strength and deformation by the form

$$\frac{C}{U} = \tanh^n d$$

and this is applicable to both healed and original concrete. As far as visual observation the behaviour of healed concrete is the same as ordinary concrete, following the same process of hardening.

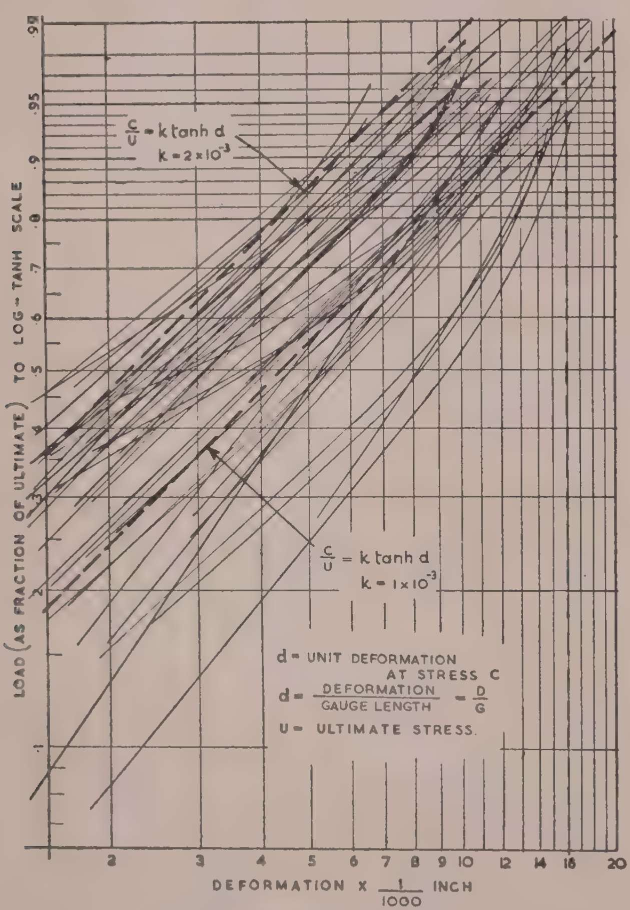

Fig. 16.—Deformations illustrating hyperbolic tangent form

Applications of Healing in Practice

The fact that concrete will heal offers possibilities on site, but the author considers that it should only be carried out under expert supervision. There is no reason why members that have cracked or failed should not be healed to enable them to carry further stresses. One of the problems is that of ascertaining the degree of healing that has taken place. It is necessary to maintain damaged parts thoroughly wet by water spray or damp hessian so that the fractures surfaces in the concrete are kept moist, and to continue the treatment for one to three months in general. The engineer should then satisfy himself that the healed section is of sufficient strength to carry on with its job of work. This would probably be by a load test. The formation of the white deposit at the ... will probably be present as an indication ... ing is taking place. The most frequent cause of f... ith green concrete in precast members. Often precast slabs, etc., are lifted or moved at too early an age with the result that failure takes place. Usually it is in the form of a tensile crack (very visible) and this extends through the unit or member being visible at the compression face, where local compression failure is present. This takes place at an age of 1 to 3 days and such members when cured are of adequate strength for their intended purpose. In such cases the moisture for healing is still contained in the green concrete. The author has seen one example of precast roof units to a farm building that had presumably suffered such failure as quite a number had

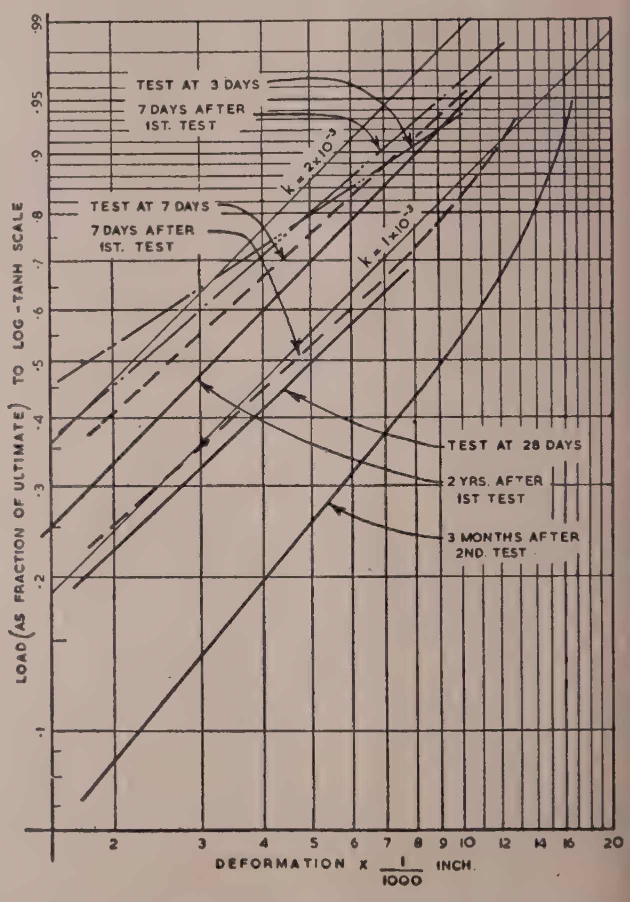

Fig. 17.—Deformations illustrating hyperbolic tangent form for particular cases

been erected and were satisfactorily in use although the cracks present in them showed clearly that they had been handled at too early an age when demoulded but must have cured themselves before the time of erection. Although this paper is primarily concerned with compression healing, another case was observed where a mould fault caused shear failure cracks at the ends of some purlins when they were stripped at 24 hours. As the section was not reinforced for shear, healing must have taken place, for when tested to destruction the end of one purlin carried 6 cwts. shear load corresponding to 50 lb. per sq. in. before failure. The other end took 4 cwts. and the second purlin 9 and 3 cwts. at its ends.

Another point where a similar process takes place is in the construction or day joint. The lapse of time between pouring one section of concrete and another is sometimes only a day or two but quite often a matter of a week or two. In extreme cases it may be months. The healing process is quite probably a contributory factor to efficient bond between old and new concrete

in such cases. It is suggested that better concrete construction would be achieved if more care and attention was paid to construction joints. The following points are important.

1. The old concrete should be thoroughly hacked to expose raw concrete, i.e., concrete below the surface and which has not been exposed to the air.

2. This surface must be thoroughly saturated with water to ensure that it is moist enough to allow the bonding (or healing if the term is used in this connection) to proceed.

3. A layer of grout should be placed against the raw concrete face and allowed to "go off" before new concrete is placed.

4. The new concrete should not be too wet and should be worked well into place and consolidated.

It may appear that this is merely stating what is given in most specifications but in the authors' opinion it is a vital point which is often glossed over and which is most difficult to achieve efficiently on site.

Acknowledgements

The author would like to express his appreciation of the assistance and guidance given by Mr. V. C. Davies, B.Sc.(Eng.), M.I.Mech.E., head of research at Battersea Polytechnic, under whose supervision the research was carried out and also of the criticism given by Professor A. L. L. Baker, of Imperial College. This paper is taken from a thesis prepared at Battersea Polytechnic, and is published by permission of London University.

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Book Reviews

Elasticity in Engineering, by Ernest E. Sechler. (New York : John Wiley, 1952 ; London : Chapman and Hall). 419 pp., 284 illustrations, 9¼ in. × 6 in. 68s.

The structural engineer is becoming more deeply involved in the stress analysis problems of shell roofs, deep beams and their panels which demand solutions based on the fundamentals of the theory of elasticity. Dr. Sechler points out that the classical reference works on this subject are those of Love, Timoshenko and others, but the publication of his book, "Elasticity in Engineering," meets the long-felt need for a work which sets out under one cover the essentials of the fundamental theories, illustrates their application and provides numerous examples.

This is an admirable, well-arranged book and should be of considerable value to both the student and the practising designer. It is efficiently annotated with references quoting book, chapter and page, a feature which in itself is of great value to student or specialist. The author's preface states that the subject-matter was largely determined by the needs of the aeronautical structural engineer, but there is no doubt that it should be of equal value to the structural engineer concerned with buildings and particularly to those who are alive to possible design economies which may be obtained from a study of the relation of panel stresses to frame stresses.

The first section establishes the fundamental equations and the analysis methods. Their application to stable structures is illustrated in the second part ; a valuable feature of this section is the comparison of the rigorous solutions with the better-known approximate solutions. The third section deals with buckling problems of steel columns, beams, plates and shells. R. J.

Foundation Engineering, by Ralph B. Peck, Walter E. Hanson, and Thomas H. Thorburn (New York : John Wiley ; London : Chapman & Hall, 1953). xix plus 410 pp., 9¼ in. × 6 in. 54s.

This book does not deal merely with the soil mechanics aspect of foundation engineering as might be expected from the background of the senior author. It is quite comprehensive in that it deals with the design of foundations from the soil survey to the structural design of the various foundation elements.

It is arranged in four parts. Part A deals in detail with soil classification, identification and properties, and would make quite a good text-book in itself. Part B is largely descriptive matter concerning foundations and the methods of construction, whilst part C deals very fully indeed with the selection of the best type of foundation for all kinds of environment and geological conditions. Finally, in part D, several chapters are devoted to the design calculations for the structural elements of a wide range of foundation types.

Although it is up-to-date there is little that is completely new in the book. But it is unique in that there has been no previous attempt to include all the information it contains in one volume.

The authors have assumed that the reader will have little prior knowledge of the subject so that the book can be used by students. On the other hand the treatment is so comprehensive that many widely experienced engineers will learn much from it.

Minor criticism can be levelled at the text. For example, the statement on page 246 that the frequency of vibrations in foundations which causes most settlement is of the order of 1200 per minute is far too sweeping. And the rejection on page 239 of more recent and more accurate pile-driving formulæ in favour of the ENGINEERING NEWS formula, which the authors admit gives factors of safety anywhere between less than 1 and 10, will not please Dr. Faber. But these are isolated examples.

The book is very readable, well illustrated, and contains a large number of worked examples. Various problems to be worked by students are given at the end, but these would have been more useful if answers had been provided.

W. E.

Soil Stabilisation in Fine Materials*

Discussion on the Paper by Mr. S. J. Crispin, M.I.Struct.E., L.R.I.B.A.

The CHAIRMAN introduced Mr. Crispin, who then presented his paper. He also exhibited two cinematograph films illustrating soil stabilisation: one (in colour) by permission of the Ministry of Works, and the other (in black and white) was shown by courtesy of Messrs. A. Monk & Co., Ltd.

Discussion

The PRESIDENT, proposing a hearty vote of thanks to Mr. Crispin for his most interesting paper and two most instructive films, said all would agree that he knew his subject thoroughly; and his description of the work shown in the films had added much to what the meeting had learned.

One had noticed, continued the President, that Mr. Crispin had not mentioned cost, and it was a most important omission. One hoped that, either at the meeting or later on, he could add some figures showing the relative costs of the methods he had described, because cost was a fundamental consideration in any engineering project such as had been discussed and illustrated. No doubt soil stabilisation was very economical, but if it were possible for him to give some detail of costs it would be very much appreciated.

(The vote of thanks was accorded with acclamation.)

Mr. K. E. CLARE (Road Research Laboratory, D.S.I.R.), said that in recent years there had been a considerable increase in the extent to which methods of soil stabilisation had been applied in road construction, particularly for pavements carrying light to medium traffic. The probability of an increase in the next few years in the amount of new construction on main roads raised the question of the extent to which stabilised soil could be used to construct heavily trafficked roads, and Mr. Crispin's contribution was timely in that respect, dealing as it did with some of the practical problems involved.

Considerable interest was taken by the Road Research Laboratory in the very large project from which the data in the paper were mainly drawn, and it was gratifying to note that some of the testing techniques which the Laboratory had helped to develop, e.g., for measuring compressive strength, density, moisture content, etc., had been found to be valuable in the control work.

With regard to the specification employed for the mixed material, queries had often been received at the Laboratory regarding the function of the bitumen emulsion employed in the soil-cement-emulsion mixture. They considered that the bitumen component can confer a useful degree of flexibility on soil-cement mixtures, such as those used at the site concerned, that owing to the granular nature of the soil would otherwise attain high strengths and be excessively rigid and liable to develop large cracks. Laboratory tests had demonstrated that a small proportion of bitumen emulsion could also effectively reduce the water absorption of a soil-cement mixture. The latter might be a useful factor in preventing frost damage to the unsurfaced soil-cement base during construction. It would be interesting to hear if the author had acquired any information that supported those views.

Two small points relating to the field work called for comment. Discussing the use of the face shovel, the author viewed with disfavour the use of that machine for excavation at right-angles to the stratification planes in soil. While he was to be commended for his emphasis on the selection of the best types of soil by horizontal working, the use of the face shovel as a method of homogenising soil should not be overlooked. In some pits in Southern England layers of gravel, sand, silt and clay alternated at close intervals through the depth of the pit, and horizontal separation was practically impossible. Further, a vertical blend of the soil types present (with the exception of top-soil) might be more suitable for stabilisation than an individual soil type.

In the section dealing with the mixing of additives the author made the somewhat surprising comment that, for light construction, hand tools might be employed. Since one of the major advantages of the mix-in-place method of processing to which he was referring lay in the economy achieved through high output as a result of mechanisation, either the lightness of the construction or the simplicity of the hand tools involved would seem to need qualification.

While the advantages claimed by the author for the pre-mix method of mixing were real and substantial, the fact that a variety of strata intersected the level of the proposed formation did not invalidate the mix-in-place method entirely. Most soils varied in texture to some extent even in one geological stratum, and mixing *in situ* only became difficult if the variation was extreme, i.e., from heavy clay to gravel.

An accomplishment on which all those concerned in the work described could be complimented was found in the ingenious methods adopted for stockpiling, batching, mixing, distributing and laying the soil and stabilised soil. All the operations were undertaken using machinery originally designed for other purposes or specially evolved on the site. The fact that the work was so successfully undertaken in the way suggested that even greater economies would be effected when the manufacturers of civil engineering machinery turned their attention to designing plant specifically for stabilised soil construction.

Mr. CRISPIN, in thanking Mr. Clare for his contribution, said it dealt with quite a few matters which were not mentioned in the paper and which were very interesting; they tied up with what he had stated.

He would not like to enter into a discussion concerning the use or mis-use of bitumen emulsion in general circumstances, but there was no doubt in some cases it served a purpose. He felt that sometimes it might not be necessary, and could be omitted. In the mix it was very essential that it should be distributed evenly, and when one took a sample from a pile one could tell in a moment if the bitumen were not distributed evenly throughout the particles. He felt quite certain that in the work described there was good distribution over the

*Read before the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1, on Thursday, March 11th, 1954. Lt.-Colonel R. F. Galbraith, M.C., B.Sc.(Eng.), M.I.Struct.E., A.M.I.C.E. (President), in the Chair. Published in THE STRUCTURAL ENGINEER, Vol. XXXII, No. 3, pp. 73-82. March 1954.

particles, and it served the useful purposes of providing elasticity and water-proofing.

With regard to the action of frost, he felt that sometimes stabilised soil could be laid when one would question the laying of concrete. However, the stabilised soil did not seem to be affected unduly by frost. The worst example of frost action experienced on the whole of the site was to the extent of about 1 in. depth at the surface, which happened to be left exposed on a frosty night ; underneath it was not affected. Bitumen seemed to be of great help against frost action. It was fortunate that the frost effect did not occur on the top layer, because it would have been difficult to put back 1 in. of stabilised soil in those circumstances.

If one were mixing strata and were looking for good and standard materials he was afraid the use of the face shovel sometimes upset one's calculations ; but there was no doubt that there were many places where the face shovel could be used to make a mixture of two strata into something that one could use.

He had mentioned hand tools in order to put everyone off the idea that it was necessary to do 14 in. work in order to stabilise soil. He had formed 4,500 yards run of 3 ft. wide path in 4 in. thick stabilised soil, all of which was placed by hand. It was in a most inaccessible position. It was not necessary to have a job of 53,000 cubic yards in order to use stabilised soil ; we could go to practically any size of job, down to that of the garden path, and use small hand tools ; it was no more impossible than the making up of a bit of concrete.

No doubt in the future there would be available machinery which was more applicable to work of that kind ; in the work described it had been necessary to use such machinery as was available and it was difficult to decide which was the best machinery to use. There were 29 of the mixers on the job at one time ; no doubt when mixers designed specifically for that work come on to the market there would be quite a different arrangement.

Replying to the President's request for information on costs, Mr. Crispin said he could not give definite figures. He felt it should be borne in mind that the work was carried out without the importation of any hard core for the sub-base or any aggregate for the stabilised soil ; if we looked at the matter in that way we could draw our own conclusions about cost. Transport, site conditions and the geographical position of the job would naturally affect costs quite a lot.

Mr. P. L. Capper (Member of Council) said the meeting had listened with very great interest to the paper on what was perhaps a rather unusual aspect of structural engineering. When we considered the matter, however, a road foundation was just as much a structure as was the foundation of a building, in that it transmitted a load to the soil ; it occurred to him that the principles which had been so successfully made use of in the big project described in the paper might quite as well be applied in some cases to the improvement of the bearing capacity of the soil under an ordinary foundation for a building or a bridge pier. It had been done and he considered there was considerable scope for development in that way. Structural engineers could gain a lot by co-operation with the road engineers in applying the principles of soil mechanics to that problem.

Commenting on the figures given by the author concerning the strength specification for soil stabilised by additives, Mr. Capper asked if he had any information about the specification for the mechanically stabilised soil as used for the formation, indicating the maximum density and the strength to be obtained in the field.

Mr. Crispin said the application of soil stabilisation was not confined to the underside of roads ; it could be put to use in very many different ways, e.g., for the foundations of buildings or the sub-strata for the foundations of buildings. He mentioned a 105-ft. diameter tank on a job at the moment ; no hard core would go underneath it and mechanical stabilising would be used to make a load carrying sub-base.

Replying to Mr. Capper's question concerning the specification for mechanically stabilised soil, he said the density of the sub-grade was 108, but no strength figure was given.

Mr. L. S. Blake, who had been privileged to inspect the site to which Mr. Crispin referred, said that the construction techniques used there were very interesting.

Soil-cement was rapidly gaining confidence in this country, as shown by the fact that more than one million square yards of base or sub-base for roads and runways had been constructed since 1945. That it was an economical method of construction, he believed could be established by its recent gain in popularity. A wide range of soils could be stabilised with cement, and even clays had been satisfactorily stabilised.

Among the points which Mr. Crispin had made in his paper was the fact that a thorough soil survey was essential ; that was very important.

Commenting on a statement in the paper that, due to the low moisture content of the soil-cement, the work could be done in colder weather than in the case of concrete, Mr. Blake felt that it was necessary to take just as great care with soil-cement as with concrete. A 6 : 1 concrete with a water/cement ratio of 0.7 contained about 10 per cent. of moisture, which was comparable to the moisture content of the soil-cement mentioned.

Mr. Crispin said it was quite a good point that the economics of soil stabilisation had been indicated by the increasing popularity of the process.

With regard to the number of soils now stabilisable, they were increasing in all directions. In places such as Kensington Gardens one could see how quickly a path was stabilised by people walking over it. The clay content of soils that could be stabilised nowadays was quite a bit higher than when the work mentioned was started. The economics of the process when using additives depended on the ability to do the work with reasonable amounts of additives, whether they be cement or other materials.

Concerning frost, he did not wish to give the impression that stabilised soil could be laid when the temperature was (say) 22° below zero, or anything like that ; he had merely wanted to show that on the job the frost did not seem to give so much trouble as when using ordinary concrete. That might be the fault of the ordinary concrete, which had a much higher moisture content than 10 per cent. or some other percentages which were sometimes mentioned. One of the troubles in putting down concrete in cold weather was that there was already too much water in it. Things might be different if we could get the concrete comparatively dry as quickly as we could the stabilised soil ; and he had stated in the paper that we could not put down stabilised soil unless we had got it to a comparatively dry state. It was his experience that stabilised soil did not seem to give so much trouble as concrete in cold weather. He might

have taken a big chance and possibly the work was done in an exceptionally good year.

Mr. ARTHUR H. LEY (Associate of Council), said he was extremely interested in the paper and was particularly impressed by the contours and profiles which Mr. Crispin had been able to achieve in his work. He asked if we should be able to obtain equally good profiles by the mix-in-place method.

Mr. CRISPIN said the only trouble with the mix-in-place method was that if one wanted a super-elevation (say), a 5 ft. 6 in. rise on a 35 ft. width, it was necessary to ensure that one had a fairly regular material which would follow that contour. If one could open up strata suitable to be mixed-in-place he did not see why one could not use that method in super-elevations or anything else, so long as one could get the formation one required in the end and its surface was not cut by sands in one place and clays in others. It all depended on the geological formation.

The biggest difficulty about mix-in-place was, as he had mentioned, the variation in strata, and there was also some little difficulty as to whether it became economical to take it off. We could mix-in-place only to a certain depth ; we could not do it to —14 in., for we should not get the density required in the bottom layer. Portions would have to be taken away and brought back after the bottom layer was in, then the apparatus for mixing-in-place had to work on a set material directly underneath it. He had experimented by mixing-in-place with the general mix, and the work was done very well. Mr. Crispin showed a sample of soil stabilised by the mix-in-place method.

Mr. G. H. GREASLEY, who welcomed Mr. Crispin's very good paper, felt that he had been all too brief in his reference to a cardinal point regarding the stabilisation of soil, and that was aeration. He asked if a little more could be said about that, because he felt it was absolutely necessary.

Mr. CRISPIN agreed that aeration was one of the methods used, and said that what he had tried to advocate was a method of achieving the correct moisture content for mechanical stabilisation. Aeration was used quite a lot in the work he had described, and it had been found that in certain circumstances it was an easier method than sub-drainage for taking out the moisture. Sub-drainage was no doubt more costly. A very large area was brought to the correct moisture content by aeration on suitable days ; it could be done even in our climate, given sunshine or a drying wind. An ordinary disc harrow had been used to expose the particle surfaces, as shown in Diagram III. There was no doubt that the taking out of moisture by aeration was of very great advantage, and it could be used not only in making a road formation, but was equally advantageous in the bottom of a drain or the bottom of a foundation.

At the same time, quite a lot could be done by drainage, even if the two methods were combined ; the free water could be got out by pre-drainage, and the capillary water around the particles, which still lubricated them, could be got out by aeration.

He had said quite a lot about it in the paper, and indeed, he was very keen about it. He agreed with Mr. Greasley that aeration was one of the finest methods for reducing the moisture content ; it was very, very useful in most cases, especially in forming sub-bases.

Mr. S. R. BRODERICK commented that the paper was well written by a practical man—a great asset when dealing with works described in the paper.

It was stated in the paper that "if the water cannot be completely removed it is only necessary to obtain a required stability during the time of construction, and allow the ground to revert to its natural conditions when sealed by the finished overlay."

He asked what would happen if there was a high degree of vibration in the pavement above—would there be disintegration of the stabilised soil?

Mr. CRISPIN said that if one had compacted the grade there was very little room for the water to get back into it. When thinking of the condition of dampness in sub-bases, etc., the work should be divided into three sections :—

Section 1, the natural ground underlying portions on which work of any kind was done; section 2, the mechanically stabilised sub-base brought to required density, and section 3, the slab material mixed with additives and set.

In section 1 the strata would be partly drained and its surface somewhat consolidated. Unless there was a very high pressure exerted on the water, this strata, when covered, reverted to its natural state and any vibration should have no more effect than it does in many natural situations where a water bearing strata carries a load carrying strata over.

Resistance to vibration by the second layer mechanically stabilised was improved because of its bearing at or near to optimum density.

The situation could not occur in the third section after setting time.

Lt.-Col. W. G. H. NORTH asked whether there was not a marked reduction in the mixing efficiency with the mix-in-place method as compared with the pre-mix method, and if so, would it not mean using higher proportions of stabilising agents to give the required strength.

Mr. CRISPIN said it would require a long answer to deal with the specification for mix-in-place stabilisation. He looked upon the pre-mix method as more suitable for certain structural requirements than the mix-in-place method. One could use the mix-in-place method, for instance, for a farm road, and one would not be tied down to a specification so rigid as that required for the job he had described, to carry 120-ton loads at 60 m.p.h. In many cases where a comparatively light load had to be carried and where there was a suitable strata the mix-in-place method would be applied, often with practically no specification at all ; the work would be done on the basis of one's judgment of the site.

Mr. F. W. HINDSON, as a representative of A. Monk and Co., Ltd., the contractors who had carried out the work from which much of the data for the paper had been collected, congratulated Mr. Crispin on having dealt so competently with the subject of stabilised soil in general, without tying himself down too much to points concerning this job only, which must be regarded as being of a special nature.

Regarding the use of bitumen emulsion which had been discussed, a further point worth considering was that the contractors considered that it had helped in the compaction of the stabilised soil, probably due to a lubricating effect on the particles. The achievement

of the specified density in stabilised soil had certainly been easier than in the case of the "raw" soil in the sub-grade.

Further to previous remarks concerning the use of the face shovel shown in the film, this machine was actually working at a borrow pit face where a considerable depth of fairly uniform soil was encountered. He felt relieved that Mr. Clare, who had, on many occasions, visited the contract shown on the film, had withheld the fearsome word "homogenising" from Mr. Crispin until now when the job was completed. Otherwise he felt it might have had considerable use in the particular job discussed.

Mr. Hindson felt that it might add to the interest of the meeting to know why the contractors had decided against the *in situ* method of mixing. It was felt at the time the job was commencing—and he was thinking back two years—that there were three main points against its adoption. Firstly, the Specification required the compaction of the sub-grade to a given density. Secondly, the 14-in. thickness of stabilised soil was specified to be laid and compacted in three layers. Thirdly, the contractors considered that they could not guarantee the specified strength requirement at all times, if *in situ* mixing was adopted, due to the known reduced mixing efficiency of this method.

Another point against the use of *in situ* mixing had come to light during the progress of the job. The line of the track from a soils point of view was not as uniform as had been anticipated, much of the soil met with at sub-grade level being unsuitable for stabilising. In point of fact, a lot of the sand for stabilising had come from borrow pits off the line of the track though still within the confines of the site.

Mr. Hindson was of the opinion that most of future stabilised soil in the county would be mixed by *in situ* methods, but on the particular job being discussed the multi-layer construction, the high strength specification and the variation in soil type had made it an uneconomic proposition.

The drying out of formation merited some mention. In one particular case where disc and chain harrowing had failed due to the wet atmosphere, lime had been tried and had failed. On another occasion, in one particularly troublesome spot cement had been ploughed in and the sub-grade "stabilised" before the actual stabilised soil was laid. It had worked very well but was, of course, an isolated instance due to expense.

With regard to requests for cost data, he felt that Mr. Crispin was wise to leave that till a later written statement. As preliminary information, the material costs could be gauged from the following : The amount of cement used was about half that required for 4 : 2 : 1 concrete, i.e., about 280 lb. per yard^3 of stabilised soil. The Bitumen Emulsion amounted to about 5.4 galls. per yd.^{3} Sand dug from borrow pits and placed behind mixers at 2-3s. per yd.^{3}, which put the cost of materials only in the 16s. to £1 per yd.^{3} range.

Mr. CRISPIN thanked Mr. Hindson for the points he had made.

On the motion of the PRESIDENT, the meeting applauded Mr. Crispin for the way in which he had answered the questions raised in the discussion.

Written Discussion

Mr. E. H. BATE writes : If a guest, who thoroughly enjoyed the above lecture, may be permitted to ask for further information, I should like to put the following questions to the author :—

1. Some interesting figures were given on average strength of test cylinders, and at one time some very high strengths were mentioned. Has the author worked out and could he give figures on the co-efficient of variation for the site core cylinders which he favours ?

2. In one point the author mentioned that he has been satisfactorily stabilising trench bottoms. Could we have some further information of method employed ?

Mr. CRISPIN replies : The co-efficient of variation has not been calculated for the work mentioned. I hope later to give further details of the tests. High figures, around 1,000 lb. per sq. inch, were not frequent but occurred in all methods of testing.

The application of stabilisation to trench bottoms and other site situations is very interesting and consists in all cases of getting the ground to the correct moisture content by sumps or other methods of water extraction and at once increasing the ground density by rolling or ramming.

Mr. D. A. CRESWELL (Associate-Member) writes : There are one or two points concerning the use of excavating plant that I would like Mr. Crispin to amplify.

Mr. Crispin considers the Dragline to have "a tendency to cause disturbance," since for efficient working, and certainly for an output of 600-700 yards a day, this machine must stand on the ground to be excavated, the only disturbed ground is that left by the bucket. Is it not possible for an efficient operator to leave this excavated formation in as good condition as would a face shovel ?

With respect to the loading of lorries by Dragline, my experience has been the opposite to Mr. Crispin's. With lorry and machine at the relative levels shown in Diagram II E, there would, I feel, be considerable waste of material. The normal operation of a dragline precludes easy filling into wagons unless the bucket is tipped when almost at jibhead. Release of the dragrope below this level allows outswing of the bucket that must cause spillage and possibly damage to the lorry. Was any special bucket or attachment used ?

On page 74 of the Journal, the face shovel is proposed for trenching, but I feel that in many respects it is very inferior to a Backactor for this work. If the face shovel straddles the trench, i.e., working forward—then there is danger in collapse of the trench wall, but the operator can bottom up clean and to level. However, if the machine works backwards on uncut safe ground then it is almost impossible to bottom up the trench due to spoil spilling over the front of the bucket. The Backactor completely overcomes this difficulty. I have not found much waste when either of these equipments is used to load lorries. Does not Mr. Crispin agree that the face shovel should be used to cut a face when standing at the bottom and a Backactor should be used standing at the top ?

Mr. CRISPIN replies : There is no doubt the efficiency of the operator is most important in all excavator work. The drag line is very useful when it is necessary to stand as far as possible from the formation, but as the long reach and throw of the bucket is increased so the irregularity of the surface tends to get worse. Worked with great care, parabolic sections were formed within small limits of finished line but the general tendency, especially in wet ground, was to leave a disturbed surface.

The saving of waste in loading depends on the relative positions of the loader and the lorry and in many situations it was easier to get the lorry into a position where the drag line would fill it without waste than to get a lorry into the necessarily close position required for loading with a face shovel.

I agree the Backactor is best for trenching in most situations. The face shovel when standing on the bottom and the Backactor when standing on top is a good general principle but must sometimes be altered because of the width of the trench, the ability to carry mechanical plant on a newly excavated bottom and the necessity to divide excavations into strata for stabilisation of future replacement.

Mr. T. N. W. Ackroyd (Associate-Member) writes: The paper presented by Mr. Crispin is a valuable one in that it gives factual description of one of the first and largest pre-mixed soil stabilisation jobs in this country. The paper raises many interesting points upon some of which the writer would like to make a few comments.

The writer cannot agree that the use of cinders is a reactionary method against the principles of soil stabilisation since soil stabilisation is a method of construction whose chief advantage is its cheapness compared with more traditional methods. The use of cinders may under certain circumstances be the best solution, and it is considered that, as a stabilising agent, they are in the same class as other admixtures. Although their stabilising action has not yet been fully investigated by laboratory tests there have been numerous examples of their great value on difficult sites (such as sticky clay), where they have formed a stabilised surface.

As regards wet formations, it has been found that with mix-in-place stabilisation successful aeration can sometimes be achieved by putting a Pulvi-mixer or other form of rotary-hoe over the site with its back hood lifted up. The pulverised soil is then sprayed into the air and consequent upon the large amount of soil area presented to the drying effects of sun and wind, the required results are rapidly achieved.

It is felt that the term "lubrication" used in connection with the action of water on sand is somewhat of a misnomer. It has been shown that water in contact with many common minerals such as quartz does not act as a lubricant. That is, the effect of water on quartz is to raise the co-efficient of static friction and not lower it. It is now generally considered that the loss of stability which may occur when a very wet soil is subject to load is due to the increase of the pore water pressure and the resulting decrease in the shearing resistance of the soil.

It has been stressed that the use of bitumen as well as cement was useful in contributing towards the elasticity of the stabilised material. It may be that with the sand used, a suitable material for stabilisation could have been formed on the lines of the "wet sand mix" so successfully developed during the war for stabilising wind blown and desert sands.

Although the writer would agree that the use of the "pre-mix" method for this job was right, he does not agree that the mix-in-place method could not have achieved similar results. Using the mix-in-place method a 14-in. finished stabilised formation could have been achieved by two layers each 7 in. deep. It has been the experience of the firm with which the writer is associated, that mix-in-place methods can conform to very rigid specifications and soil stabilised by such method can be successfully subjected to heavy loads. At least one airfield in this country, which has been stabilised by the mix-in-place method, is used by heavy aircraft.

There would appear to be little reason why the accuracy of the slopes and gradients obtained by the mix-in-place method should not be the same as that obtained by the pre-mix method. Similar plant—dozers, graders, rollers (wobbly-wheel and smooth) are often used on both types of jobs to produce the final finished shape.

The real value of soil stabilisation would appear to be that it produces a finished pavement which is as good as, as efficient, and, in some respects, better than concrete as a base for subsequent carpeting, and, at the same time it is cheaper and quicker to construct. Cheapness is achieved by mechanisation and by a reduction in, or an absence, of the cost of materials, such as hardcore or aggregate.

The use of stabilisation for highly trafficked roads, tracks and paths is economic when the amount to be done is such that cheapness can be achieved by efficient mechanisation. Although the intention of the author to show that stabilisation can be done using hand tools is laudable it is doubtful whether such a method is really satisfactory except, perhaps, under certain circumstances; for example, some eight years ago the writer constructed a very cheap path by stabilisation, using only hand tools plus a garden roller. Cheapness was achieved because the only material which had been paid for was cement, the soil on the site (his back garden) being utilised. He doubts very much, however, whether this simple method of construction has any wider applications.

With reference to the tests for soil stabilisation (described in B.S. 1924) it has been the writer's experience that the "Speedy" method of measuring moisture content is excellent for use with sand but is probably not so accurate for gravels and is difficult to apply to clayey soils.

Mr. Crispin replies: The use of cinders in a formation is a method of stabilisation in many cases but whilst economic at or near industrial areas it becomes very costly, in fact impossible, in many situations and the more intensive study and use of local materials in such areas will allow work to be done giving the same results as winders or hardcore in other situations.

The aeration of soil by the Pulvi-mixer or other rotary plant is possible but it may require several passes if the evaporation period is confined to the time the particles fly through the air. Aeration can be applied to the mix-in-place method with great advantage but the restriction of depth applies in many cases.

I still consider that surface water on the particles in the work referred to and in many others acts as a "lubricant." In theory it may not do so with quartz but in practice the compaction of, say, the residue of china clay works, which is quartz, whilst in a surface wet condition, would present great difficulties.

The use of bitumen in this work has attained the desired result but its inclusion in all cases may not be necessary, in some cases, however, it may be the only binding material.

The mix-in-place could do a 14 in. layer but not so economically or with the standard of control throughout as the pre-mix did in the case mentioned. I think a 7 in. layer is bordering on the line where correct density throughout can be obtained, working in two layers by mix-in-place will mean more handling, out and back, and it becomes practically a pre-mix method. The wearing away of tines in the rotor must also be considered when laying a fresh layer on one already set underneath. Mix-in-place has a multitude of uses but cannot be of the greatest advantage in all cases.

Generally I am in agreement with Mr. Ackroyd in other points and I do not advocate the "garden-path method" for heavy load carrying situations for, like all other methods, it has its limitations.

Prestressed Steel Lattice Girders*

Discussion on the Paper by Mr. R. A. Sefton Jenkins, B.Sc., A.C.G.I., A.M.I.C.E., A.M.I.Struct.E.

Mr. SEFTON JENKINS, presenting his paper, said he believed that Professor Magnel's paper on prestressed steel, published in 1950, was the first that had been published on the subject. Professor Magnel had said that he would very much have liked to be present at the meeting, but he had been to America and would not get back to Belgium until the following day. Mr. Jenkins was grateful to him for having sent a photograph and some figures for his use.

The photograph (Fig. 13) shows an aeroplane hangar at Melsbroek, and consists of secondary beams spanning 49.000 m. (161 ft. approx.) from columns at the rear on to a main prestressed beam and cantilevering beyond the main beam a further 17.000 m. (56 ft.) to the front of the hangar. The main girder, which is continuous over a central column, consists of two spans each of 76.5 m. (251 ft.) and is prestressed by two sets of prestressing wires passing along the bottom chord at the outer ends of the beam and up over the central support.

Fig. 13

Mr. Samuely, in Great Britain, had built some structures which were prestressed, and perhaps in the discussion he would say something about them.

Mr. Jenkins explained the principles of prestressed steel and the reasons for carrying it out and explained that he had been working on a slightly different approach to the problem from Prof. Magnel and Mr. Samuely, that of prestressing outside the depth of the girder.

He showed a photograph (Fig. 14) of a prestressed steel structure based on this principle where he had been responsible for the design. This is a new factory built by the Harlow Development Corporation for occupation by Standard Telephones & Cables, Ltd.

Mr. Jenkins went on to show a hypothetical girder that had been designed by Prof. Magnel firstly as an unprestressed girder and then as prestressed girder with the prestressing within the depth of the girder. From a comparison of these two designs Prof. Magnel had arrived at the savings shown in Fig. 15. Mr. Jenkins had taken Prof. Magnel's figures (with his very kind permission) and had redesigned the girder prestressing it outside the girder but keeping to the same overall depth of construction loads, etc. The result is shown in Fig. 16.

Discussion

The PRESIDENT, proposing a very cordial vote of thanks to Mr. Sefton Jenkins for his paper and for his presentation of it, said the Institution was greatly indebted to him, and particularly for the comparison of costs which he had given.

Fig. 14

One wondered whether the costs in Britain worked out the same as in Belgium. One had rather a feeling that some of the steel manufacturers here might have something to say about the savings of weight of steel and the decrease of labour costs to be effected by prestressing, and one hoped Mr. Sefton Jenkins would confirm that those savings did in fact materialise in Great Britain.

(The vote of thanks was heartily accorded).

Mr. F. J. SAMUELY (Member) said he felt that the reference to prestressing within a girder was possibly

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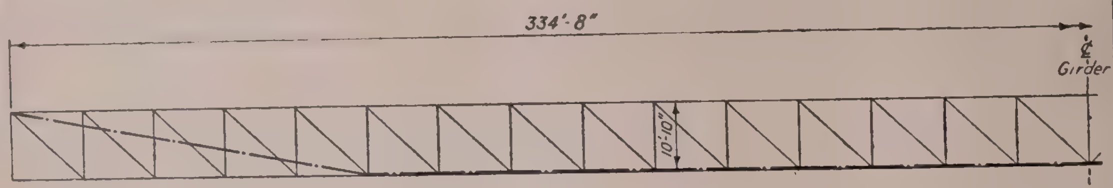
*Read before a Meeting of the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1, on Thursday, February 25th, 1954. The President Lt.-Colonel R. F. Galbraith, M.C., B.Sc.(Eng.), M.I.Struct.E., A.M.I.C.E., in the Chair. Published in THE STRUCTURAL ENGINEER, Vol. XXXII, No. 2, pp. 43-48.

slightly unfair to steel construction in general. The point was that if one had to deal with a girder that acted as a two-hinged frame, i.e., of normal steel construction, and if one stressed the tie member, the total depth of the girder was to be taken to include the tie. Prestressing in that case was done on the tie member of the two-hinged frame, and in the other case on the lower chord of an ordinary lattice girder. The point was obviously that, the lower the prestressing, the more effective it was. He asked if Mr. Jenkins had compared Professor

the large extension, the high tensile tie member could not have been effective. In order to make it effective it had to be prestressed ; this principle was exactly similar to that of prestressed concrete.

Mr. Samuely then illustrated the construction of two roofs, the first being of the dimensions 90 ft. × 90 ft., and the other 78 ft. × 81 ft. for the workshops of a college in Wigan. The whole of the area covered had to be kept free of columns, and that was done by having eight inclined trusses. Most of it was of tubular con-



ELEVATION OF PRESTRESSED LATTICE GIRDER.

INFORMATION

1................ Weight of prestressed girder.................................. {........24·3 tons M S / 0·885 tons H.T.Steel.

2................ Weight of unprestressed girder.................................. ....32·9 tons M.S.

3................ Saving in weight of inside prestressing compared with conventional unprestressed lattice.............................. 26%

4................ Saving in cost of inside prestressing compared with conventional unprestressed lattice.............................. 18%
(This assumes that the cost of H.T. Steel including prestressing is 3x the cost of M.S. erected.)

Fig. 15

Magnel's example of prestressing the lower chord with an example of his prestressing, using the same overall depth in both cases.

Illustrating some of his own examples, Mr. Samuely aid we must appreciate that reference to any prestressed aterial, such as prestressed steel, prestressed concrete, ., was slightly misleading. "Prestressing" in every e meant that the forces, by means of prestressing, re taken from where they might normally occur to her form of contruction. In his examples the forces taken out of mild steel and put into high tensile es. If the two examples of lattice construction hich Mr. Jenkins had shown had been done in mild teel, using high tensile steel for the tie member without prestressing, it would have been found that, because of

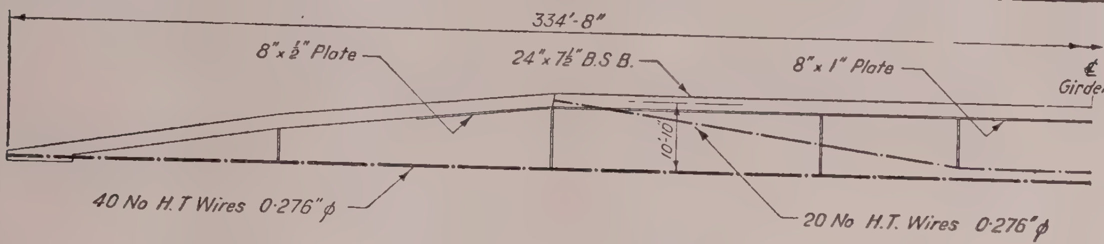
struction, and the prestressing consisted of putting the forces on to the cable by means of a number of jacks in order to make the cable a carrying member. We all know that a suspension bridge was a particularly economic structure because the cables carried forces, in the cheapest possible way. But in many cases we could not use suspension bridge construction, mostly because of the large height required, and if we were trying to arrange the suspension cable in the depth of the girder, the cable would take very little of the forces and its inclusion would not make much difference ; but by using jacks to stress the cable and forcing the whole load or an arbitrary proportion on to it, the cable could be made to work properly. The example shown was not really the best one, however, because the cable could not extend

for the full length of the roof because of the hipped construction.

Giving some cost figures, Mr. Samuely said that in the case of the roof 90 ft. × 90 ft., the total cost of the steel was £2,000 and, when the costs of stressing and erection were added, it came to £2,400. That meant that for an area of 8,100 sq. ft. the cost was just over 5s. per sq. ft., which he considered quite reasonable. The roof was originally designed without prestressing, but on the same principle of girder steel in the sloping roof, and it

Mr. Samuely showed other slides, including one of a roof, 90 ft. × 90 ft., the erection of which occupied less than a week.

Speaking of the Skylon at the Festival of Britain Exhibition in 1951, he said he believed it was the first prestressed steel structure in England; it would have been impossible to erect it but for prestressing. The whole idea was that all the cables had to be, not merely taut, but loaded with very considerable forces in order to ensure that the Skylon would not sway in the wind;



ELEVATION OF PRESTRESSED STEEL GIRDER FOR HANGER.

INFORMATION.

1.................. Girders at 8'–3" centres

2.................. Loading:– (a) Wind bracing & light weight slabs .... 21·4 lbs/□'
(b) Own self weight of girder .... lbs/□'
(c) Snow load .... 7·14 lbs/□'

3.................. Weight of prestressed girder .... { 17·47 tons M.S. / 1·60 tons H.T. Steel.

4.................. Weight of conventional unprestressed tubular & T lattice girder .... 32·9 tons.

5.................. Weight of Magnel type tubular & T prestressed lattice girder .... { 24·3 tons M.S. / 0·885 tons H.T Steel.

6.................. Saving in weight of outside prestressing compared with conventional unprestressed lattice 42%

7.................. Saving in cost of outside prestressing compared with conventional unprestressed lattice 41%

(This assumes that (a) the cost of H.T Steel including prestressing is 3x the cost of M.S. erected.
(b) the cost of girder construction from B S B's is 90% of the cost of lattice from tubes & T's)

Fig. 16

[w]ould then have cost about £3,000. So that there was a [2]0 per cent. saving as the result of prestressing.

The total cost of prestressing the first roof, which [w]as done by the general contractors, came to £100. In [th]e case of the second roof, which was slightly smaller [th]an the first, the cost of stressing carried out by the [sa]me contractor was only £25; by that time the con[tr]actors were familiar with the process.

The next illustration showed one of the roofs, with the [ca]bles; it differed from the former slide because the [sh]ape of the cable was altered at the last moment. [In]stead of jacking at six points they had jacked only [at] two points, for the results were found to be the same; [it] was not necessary to have the cables in the parabolic [fo]rm, but in the simple form of a trapezium.

the wind forces could be very large, and without prestressing it would have been very difficult to deal with them. The Skylon was actually stressed by jacking up the base of the three pylons. There were six cables, three of them attached to the structure at half the total height, and the other three at the bottom; by raising the pylons stress was introduced into the cables. In the lower half of the Skylon there was a stress of 220 tons. There was deformation in a high wind, but it was very small. The greatest possible deformation in a gale was calculated to be 6 inches, the actual deformation was smaller.

He mentioned the Skylon particularly because it was very interesting how prestressing of the steel made for much greater stability. It was so successful that it

was not necessary to guy the top of the pylons in all directions. Each guy rope was in a radial plane through the Skylon, and one of the pylons, and there was no need to do anything about holding the pylons sideways.

He added that, in order to avoid depending entirely on theory, tests were made in a wind tunnel, which were entirely satisfactory.

An illustration showed the hooks at the bottom of the Skylon, to which cables were fastened.

(See also paper published in Part I of the Proceedings of the Institution of Civil Engineers, July, 1952).

Mr. Sefton Jenkins said that all present would be indebted to Mr. Samuely for having shown his examples, and he apologised for having forgotten to include a reference to the Skylon in the paper.

With regard to girder depth, he said he believed his was a shade shallower than that designed by Professor Magnel; the centre lines were exactly the same, and there was not very much difference in the overall depths.

He was not sure that he agreed altogether that it was quite the same thing to prestress over or under the girder. It was quite true that it was a two-pin truss or arch prestressed, but if the depth were kept the same we could put more load on to the high tensile wires than we could if we prestressed in the depth of the girder; so that we could put more load into the wires more cheaply than into mild steel, and effect economy in that way.

Mr. K. J. Sommerfeld expressed his indebtedness to Mr. Sefton Jenkins and Mr. Samuely for having designed those first permanent prestressed steel structures, which in both cases were carried out by Sommerfelds, Ltd.

Discussing one or two points in the paper which perhaps needed a little clarification, he said the author had given a fairly full description of the method of manufacture of the very light high tensile steel latticed box girders. What interested him particularly was the node point, and he exhibited a sketch (Fig. 17) which he believed Mr. Sefton Jenkins had not seen so far.

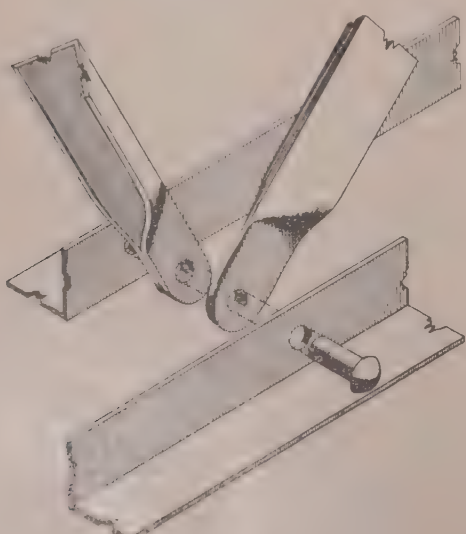


Fig. 17

The great advantage of the diagonals in the form of angles, he said, could be taken fully into account and there was no outstanding leg in the ordinary sense, or the centre line and the centre of gravity coincided, and it could be seen that all the forces met at one point. He had made a considerable number of tests and had found that small struts, of dimensions only $1\frac{1}{2}$ in. $\times$ $1\frac{1}{2}$ in., and having a length of 30 in. or so, had withstood 6 tons before buckling, with one hole on the one side of the angle; by merely pinching the legs together, however, they had given twice the strength, i.e., they had withstood 12 tons. There was a further advantage. With the very light high tensile steel sections some difficulty was generally experienced in the detailing for stress forces, the bearing areas being so small, but by pinching them together there were two thicknesses at the joint. Sometimes even the two thicknesses of the lower boom did not give sufficient bearing area, in which case *that* particular point was thickened in a mechanical way.

Mr. Sommerfeld was very interested in the slide comparing Professor Magnel's construction with that of Mr. Sefton Jenkins with the two sets of tie rods below. He felt that that kind of construction had a very great future, and he supported Mr. Sefton Jenkins' view that the total fabricating costs of that particular construction ought to be less than those of a similar construction with a lattice assembly prestressed. By and large, the fabricating costs might be proportional to the number of node points. Those node points cost about the same amount of money, and it would have been seen that Professor Magnel's drawing had a large number of node points compared with that of Mr. Sefton Jenkins with very much fewer node points.

Commenting on the President's remarks concerning increased labour cost, Mr. Sommerfeld said that undoubtedly there was some increase; but, as had been shown by Mr. Samuely's experience, the extra cost was very small indeed. There was a certain extra cost in connection with the details of the structure in order to apply the prestressing forces. But it was interesting to note that the latest structure on which his firm were working, which covered a rather large area, involved a total cost of about 7s. per sq. ft., including the cladding, a reasonable amount of lighting and a certain number of sliding doors, etc., for some storage tanks. That was the lowest figure they had achieved yet in any of their structures, prestressed or otherwise.

Mr. J. A. Williams (Member), having been concerned recently with some prestressed steel structures, said that inasmuch as they were rather more crude than those described at the meeting so far, he mentioned them with some diffidence; but they might be of interest nevertheless.

The problem had been to devise a fendering system to deal with the large super tankers now being built, having displacements of up to 60,000 tons. Describing how it was solved, he said that about 60 high tensile steel raking piles, in a single line in plan, were driven to penetrations of up to 40 ft. At one site broad flanged beams had been used, and at another some German piles. On top of the piles a heavy concrete block roughly 8 ft. square, had been cast which, during casting deflected forward the line of piles, and that constituted the prestressing. In plan they thus had a concrete beam about 240 ft. long, supported on the 60 piles. It was prestressed forward and the simple idea was that, when a vessel came alongside, the structure was deflected backwards, the energy of impact being absorbed by the bending of the high tensile piles.

Her Majesty's ship *Gothic* was the first to come alongside such a fender, and her contact had been so gentle that the structure hardly deflected. Tankers had since come alongside, and so far had been brought to rest with only moderate deflections.

During construction the concrete was cast in sections leaving temporary gaps of about 5 ft. at intervals. A

the individual blocks were cast, they came forward to ±10 per cent. of the extent calculated, but probably because pile driving is not an exact science, they all came forward by rather different amounts and had to be pulled together to line up when the gaps were finally concreted. Consequently, the final forward deflection was a little more than was calculated, but not to an

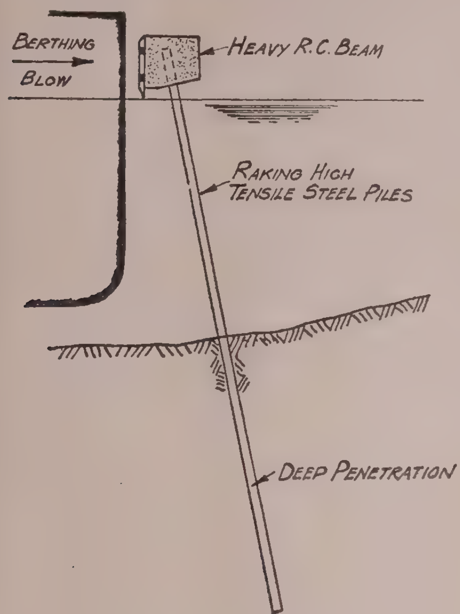


Fig. 18

extent that was in any way serious. Otherwise the construction went ahead without any great difficulty, and he could only say that, so far, the structures were behaving satisfactorily.

Coming to the paper, he referred to a lattice girder in section, and said that the author stated that if one applied a force to one flange, nothing happened to the other flange. He could not see why, because in the case of an eccentrically loaded column, if one put a load on one flange there was presumably tension on the other.

Finally, he said that perhaps it was not very chivalrous to criticise a new idea, but he was accustomed to dealing with somewhat heavier steel members, and he could not help feeling that some of the light roof trusses shown would suffer serious corrosion, possibly giving rise to high maintenance costs.

Mr. SEFTON JENKINS said there were more ways of killing a cat than by knocking it against a fender! If one had a square or rectangular section column eccentrically loaded, then, as Mr. Mr. Williams had stated, tension would be developed on the opposite side to which it was loaded. If. however, the column consisted of two chords joined by lattice members, then, provided all the joints were pinjointed, the compression of one chord would not induce tension in the other chord.

Professor J. A. L. MATHESON (Delegate Member of Council), after congratulating Mr. Sefton Jenkins on an extremely interesting account of his work, said that at the Department of Engineering in the University of Manchester they had been taking great interest in steel economy from the theoretical point of view and he would like to make one or two points as the result of what had been learned, because they had a bearing on what had been said at the meeting.

If a pin-jointed framework was considered as a device for connecting together a number of joints whose positions in space were prescribed, then it could be proved* that a statically determinate structure was always lighter than an indeterminate one for the same loading and maximum stresses.

This fact raised the question of the reason for the economy achieved by Mr. Jenkins. It might be that the explanation lay in the difference between economy of weight of material and economy of cost, and that if the roof had been designed as a determinate structure in high-tensile steel it would have been still more economical, at least in terms of weight.

In comparing the economy of two structures it was important to ensure that they were strictly comparable ; in this case it seemed important to discover whether the saving arose from the prestressing, from the introduction of material with a higher working stress or from the location of the ties below the bottom chord of the girder, which really altered the configuration of the structure.

In conclusion, Professor Matheson asked for some clarification of the symbols used in Appendix A ; in reply, Mr. Sefton Jenkins explained that in the first two equations of the Appendix, $a, b \ldots f$ were non-dimensional coefficients. R_1 and R_2 were the tensions in the ties shown in Figs. 6 and 8 and P_1 and P_2 were constants. The remaining symbols were explained in Fig. 10.

Mr. R. K. LIVESLEY wrote : Professor Matheson has mentioned a theorem on the minimum weight design of pin-jointed frames. This theorem holds if :—

(1) The positions of the joints of the framework are prescribed, and no other joints are introduced by the designer.

(2) The load system (assumed to act at the joints) is fixed.

(3) The stress criterion for each member is that of maximum tensile or compressive stress. These stresses need not be equal, and may vary for different members. They must however be independent of the member cross-sectional area or moment of inertia.

Under these somewhat restrictive conditions it can be proved that the frame of lightest weight which will support the loads is always a determinate one. A proof has been given by J. Drymael,[1] while Sved[2] has pointed out that the theorem still holds when prestressing is allowed.

There are many ways of arranging the members of a determinate frame to connect a given set of joints, and these various frames will in general have different weights. If the design chosen is not in fact the one of minimum weight, then the addition of a redundant member, prestressed or otherwise, may well result in a more economical structure. We have an example of this in the frame shown in Fig. 5. Mr. Jenkins shows that the introduction of a horizontal tie produces a lighter frame, but this is due to the original design not being the lightest possible one. If it had been, prestressing would have been actually harmful.

This argument also implies that an even lighter frame than the one shown in Fig. 5 could be achieved by reverting to a determinate structure. Admittedly, the theorem mentioned above gives no clue as to the configuration of the minimum weight structure, but merely states that it exists.

The additional prestressing shown in Fig. 6 introduces a new element. The frame has now two new joints, and the theorem must be applied to the whole frame, including these. Once again, however, the theorem tells us that an even lighter determinate frame could be

*See contribution to the discussion by Mr. R. K. Livesley.

constructed—probably retaining the new members (i.e., the tensioning wires).

Another interesting problem presents itself in the positioning of the two new joints. It seems likely that there is an optimum position for these, and it would be useful to have a mathematical analysis of this question.

The points raised above are admittedly somewhat academic. The theorem on which the argument is based has not yet been proved for a structure called upon to carry several alternative loading systems, nor for structures in which buckling must be considered. It may be that in these more practical cases a prestressed redundant frame will turn out to be the lightest.

References

[1]J. Drymael. JOURNAL ROY. AERO. SOC. Vol. 46 (1942).
[2]G. Sved. Unpublished discussion of a paper by A. J. Francis. AUSTRALIAN JOURNAL APPL. SCIENCE. Vol. 4, No. 2 (1953).

Reply by Mr. SEFTON JENKINS to both Professor Matheson and Mr. Livesley : The theorem referred to is slightly academic and although it may on occasion have application it is difficult to think of any. It means in effect that the loading must never change, and that every member in the structure must be designed to take exactly the load in that member. It means for instance that in a lattice girder every diagonal and every boom must be different from its neighbour. For purely economic reasons connected with fabrications it is obviously desirable that a girder should be as nearly the same along its length as possible.

If one has, say, an R.S.J. acting as a simply supported beam under uniform loading, then the beam will be working at its maximum stress at only one single point along its length, i.e., at the centre. If now it is possible to increase the number of points at which maximum stress occurs then the beam will be used more economically. This is what in effect is assumed with designs on the plastic theory. It should be noted that the ultimate failing load of a prestressed or unprestressed structure will be identical, the difference lies in the deformations and stresses in the elastic range. It is possible by the judicious prestressing of a structure to increase the number of points where the full maximum elastic stress is realised, when compared with an identical unprestressed structure.

Mr. A. GOLDSTEIN (Associate-Member) added his congratulations to Mr. Sefton Jenkins for his paper on an interesting form of structure.

Considering the fundamental structural principles involved he agreed with the author that prestressing below the flange was not the same thing as prestressing within the section, but certain basic factors, which apply to the more usual internally prestressed members but do not apply to externally prestressed members, must be noted. For example, there was the question of stress variations in the tensioning elements. If the prestressing element of a member is within the section—in the case of concrete—or within and attached to the bottom flange, in the case of steel, then after prestressing and relaxation to working stress the variation of stress in the prestressing element is extremely small. Generally, this variation does not exceed about 5 per cent., as had been shown by Professor Magnel in his book. However, in an externally prestressed structure such as described by the author, the stress variation in the prestressing element was very large. Mr. Goldstein drew attention to the statement in Appendix B of the paper that the ends of the girder moved apart some $\frac{3}{8}$ in, under load, and he calculated that to be equivalent to some 7 tons per sq. in. stress. Mild steel bars were used with a working stress of, he imagined, about 10 tons per sq. in. ; so that the 7 tons represented some 70 per cent. stress variation. Quite apart from the usual, and useful, automatic test load on the prestressing element, which was not present in this instance, the question of fatigue would have to be given more serious consideration if the stress variation were so high. Perhaps Mr. Sefton Jenkins could give the actual working stresses of the various components of the structure.

Another point was the question of buckling. An internal prestressing element could not cause buckling. In fact, such an element acts as a stabilising force on the structure. An external prestressing element, however, just like any other external force, could cause buckling and special provisions would have to be made to counteract this where slender sections are used. That important distinction should be borne in mind.

Creep was a problem that had to be studied very thoroughly in the case of prestressed concrete. Whilst he did not suggest that a steel lattice girder would be affected adversely by any possible creep in the steel, he asked whether "creep" could not be stimulated by rivet slip or bolt slip. Indeed, in the first girders this had been the case. He wondered whether any further observations had been made on the girders after the erection to determine whether that such "creep" had occurred because with a mild steel tie very little creep was needed—in view of the small extensions induced due to the small induced stress—to produce large losses of prestress.

Concerning the "lack of fit" method used in the calculations, he asked whether calculations based on a tied arch or tied portal with elastically yielding ends might not have been a little more direct.

The site test was one of the most valuable matters described in the paper, and Mr. Goldstein asked why the test load was limited to the design load only. For an unusual structure it would have been extremely interesting if the tests could have been taken to a post design stage, say $1\frac{1}{2}$ times the design load. He did not suggest that it should be necessary to test to destruction since clearly that would be rather expensive. Referring to Fig. 12, which compared the calculated and measured loads, he said Mr. Sefton Jenkins had stated that the agreement was quite good. Considering the bottom curve, however, and noting that the intercept between the thick lines was the measured load and the intercept between the fine lines the calculated load, it could be seen that at a point three squares to the left of the right-hand side of the graph the measured load was about three times the calculated load. Obviously there was some explanation for that, but Mr. Goldstein considered that this was not normally regarded as good agreement. It was a further reason why the test loading at post design stages would have been of interest ; certain stress redistribution might have occurred or other factors become apparent and extremely valuable data could thus have been obtained.

On the question of costs, since the contract was awarded on the basis of "design *and construct*," it was difficult to correlate the success of the tender with any inherent economy of the design and, in particular, this type of structure. Perhaps Mr. Sefton Jenkins could give two figures, which would enable comparison to be made with more orthodox construction. Firstly, what was the total amount of steel used in the girder and tie per sq. ft. of area ; and secondly, what was the "all in" cost per ton of that steel, including all labour necessary for completion and all materials.

Mr. SEFTON JENKINS, dealing with the question of stress variation in the bar, agreed that there was very

much more variation of stress in the prestressing device with prestressed steel when compared with prestressed concrete, but the variation of stress under working conditions was not as large as Mr. Goldstein estimated. The variation under application of live load was of the order of 30 per cent. The $\frac{3}{8}$-in. movement referred to was under application of complete load (dead load plus live load) of the test girder. This was, of course, a variation that occurred only once.

With regard to fatigue, it would appear from the large amount of research that had been undertaken on cold drawn wires that provided the stress was kept rather lower than in prestressed concrete, say 45-50 T/sq. in., then there was no danger.

Similarly in this stress range the stress loss due to creep would be under 2 per cent. In this connection Professor Magnel had stated that with the riveted lattice girder shown in Fig. 13 that he allowed for a possible relaxation due to combined rivet slip and wire relaxation of 9 per cent., at the same time saying that he thought this rather pessimistic and that the probable losses would be less. With a welded R.S.J. type of prestressed structure Mr. Jenkins would suggest that a possible maximum relaxation of 5 per cent. would be a safe figure.

Regarding weights of material and costs. In the case of Harlow, the total weight might seem comparatively high as there was a large amount of steel in monitor frame, glazing rails, side sheeting rails, etc., which had little to do with the problem under discussion. If the secondary beams had been placed at 7 ft. centres across the span so that the glazing or decking could be placed direct on to these, the weight of steel would have been as follows :—

1. Original design.

(See Fig. 8, February, 1954, STRUCTURAL ENGINEER).

	Wt./sq. ft.	Cost/sq. ft. @ £110 per ton
(a) Main beams ...	1.44 lb./sq. ft.	1/5d.
(b) Secondary beams ...	1.28 ,,	1/3d.
Total weight ...	2.72	2/8d.

2. Revised design.

(See Fig. 9, February, 1954, STRUCTURAL ENGINEER).

	Wt./sq. ft.	Cost/sq. ft. @ £110 per ton
(a) Main beams ...	2.12 lb./sq. ft.	2/1d.
(b) Secondary beams ...	1.28 ,,	1/3d.
Total weight ...	3.40 ,,	3/4d.
Reduction in wt. if H.T. cables had been used in lieu of bars ...	0.48 lb./sq. ft.	

Mr. GOLDSTEIN asked for the cost per ton of steel.

Mr. K. J. SOMMERFELD said he had not the exact gure, but the cost per ton of the structure was of the rder of £110 or £115. The members were very light.

Raising the question of the future method of construction of prestressed steel structures, he said his firm had ompleted three or four such buildings and, naturally, ad learned a little about the problem. They had also repared some prestressed concrete members, using the eel by the usual Samuely composite method, i.e., with e concrete on the top in compression and the steel in ension. From all those various experiences to date he ad concluded that they had not manufactured those ings in the best possible manner. He could foresee the future the use of a method of prestressing which he had recently developed ; he could not give figures, either precise or otherwise, but he was satisfied that it would show advantage in respect of price over the types of construction of which we had heard so far. The method reminded one of how, as boys, we played with bows and arrows. It involved taking a straight section (say), a steel joist, and applying forces to it in the factory so that it would bend, like a bow, and whilst it was in the stressed condition attaching a straight tie bar to the two ends. The jack or mechanism used to bend the bow, or the tie by itself, would produce both elastic and plastic bending. Thereby also a good run off would be achieved on either side of the roof.

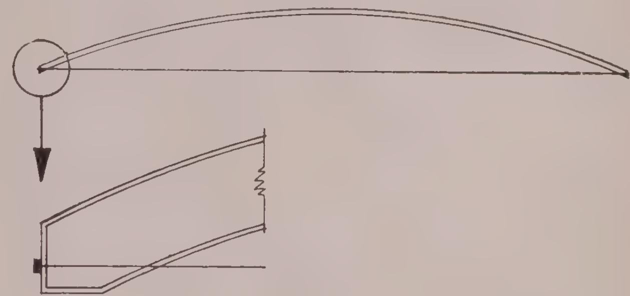


Fig. 19

The bow roof, of corrugated steel, would then be surfaced.

A criticism which had been raised by Mr. Williams against the light steel constructions was that they might be prone to corrosion. They would not be any more or less prone to corrosion than were the heavy constructions. Presumably what Mr. Williams meant was that if one allowed (say) $\frac{1}{8}$-in. of pitting, that would be very much more serious if the structure were of $\frac{1}{8}$-in. sections than if it were of $\frac{3}{8}$-in. sections. That was obvious ; but all would agree that it would be bad practice to permit a structure to rust to that extent. Furthermore, experience had shown that we could quite satisfactorily prevent that corrosion, especially for structures we had seen so far, which, after all, were covered on the top.

That led Mr. Sommerfeld to comment that far too little use was being made of one of the best media for protection, and that was bitumen. He did not mean just a thin deposit of bitumen, but an extensive coating, a coating which was stoved in a factory so that it formed a hard surface and did not scratch. Ordinary motor car and lorry wings, for example, were protected in that manner. Further, it was the habit of his firm, where structures were open to wind and weather, to put a certain amount of aluminium powder into the bituminous compounds. Aluminium being lighter, it formed on top of the bitumen. That method produced one of the most ideal corrosion protection coatings, if applied on a clean undercoat. We knew that bitumen itself would stand up for thousands of years, provided we could keep off ultra-violet rays ; the function of the aluminium powder was to prevent the ultra-violet rays striking the bitumen.

Mr. S. M. REISSER (Member), recalling a statement by Mr. Sefton Jenkins that high tensile steel was unweldable, thought that it should be qualified—because he presumed that it referred to the high tensile steel prestressing wire and not to high tensile structural steel in general. Many types of the latter were weldable and, in fact, were being used on an increasing scale in normal structural work.

Secondly, Mr. Jenkins had referred to the considerable saving in weight in a building incorporating compound

beams : were the latter of welded or riveted construction? He asked the question because the welding of compound beams always resulted in a saving in weight and Mr. Jenkins' figure could therefore be still greater if the beams to which he referred were of riveted design.

Finally, he was surprised to hear of troubles due to slip in bolted and riveted connections—which could have been easily avoided simply by using welded joints which are not subject to slip !

Mr. Sefton Jenkins said he *had* been referring to cold drawn high tensile wires with an ultimate stress of the order of 100-110 tons per sq. in.

The girder he had shown in Fig. 16 was, Mr. Reisser would be glad to hear, intended to be welded.

Mr. J. C. H. Finlinson (Associate-Member), referring to the question of buckling already mentioned by Mr. Goldstein, said he was aware that a prestressing cable within the depth of the girder could be used to control buckling, but he was not clear that this was also the case when the cable was below the girder.

If he had understood correctly, the author was proposing a girder of prestressed 24 in. $\times$ $7\frac{1}{2}$ in. rolled steel joist (plated) on a span of 334 feet. He realised that such a girder could be prevented from buckling sideways by, say, lattice purlins, but he was not satisfied about the prevention of buckling in the vertical direction and asked whether the prestressing cables could be made to prevent such buckling, and whether the author would rely on them alone.

Mr. Sefton Jenkins said that with a girder connected by a tie across the bottom, and with struts as shown, any tendency to buckling would merely move the prestressing wires in exactly the same way as when the prestressing is within the depth of the girder, where the prestressing wires are coupled direct to the girder instead of through struts.

Mr. Goldstein said the point he had made was that internal prestressing could not cause buckling. For example, the buckling load of a cylindrical tube loaded by means of a central wire tensioned and anchored at the ends of the tube and attached to the tube at one point only at the centre of its length, was four times the *Euler* load. In the case of an internally prestressed member where the number of connections between the prestressing element and the member were virtually infinite, then the buckling load is infinite too, in other words, failure would occur by crushing rather than by buckling. The well-known magician's wand, made up of tubular sections and connected by an internal string, was a simple demonstration of this phenomenon. An external prestressing force on the other hand *could* cause buckling, and whilst there were admittedly measures that could be taken to prevent buckling that was the point that he had made, i.e., these measures were necessary.

Mr. Sefton Jenkins agreed and said care must be taken in the erection procedure to ensure that the girder is not prestressed say, without both the secondary beams to stop sideways buckling and also struts between the prestressing wires and the beam or some method of ensuring that the girder did not buckle sideways.

Mr. R. J. Wilkins (Member) was particularly interested in the loading test and he congratulated Mr. Sefton Jenkins on Fig. 12, showing the comparison of the calculated and measured stresses. He asked what was the horizontal scale for Fig. 12 ; and at how many points across the flange the stresses were measured. He would like the author to confirm that the girder tested was the original one, for he was not quite clear about that.

There had been reference to holes being drilled in the bar ; the author had talked rather of pop marks, and one felt that that was a better description. The holes were very, very small indeed, and there was no question of removing much material from the steel itself.

About the back bolts, he said all elastic calculations on displacement were based on the elastic operation of the complete medium being tested, and where there were black bolts in clearance holes one was not surprised that the author had got his first excessive deflection of about 3 in.

The method of using light angles which had been described by Mr. Sommerfeld was a very nice one. Normally we riveted them on one flange, and a tie angle would then go into a state of compression stress on the edge of the outstanding flange of about 3 tons per sq. in. It seemed that in this case we were using angles in an educated manner.

Mr. Sefton Jenkins, replying to the question concerning the scale of the diagram, said it extended to halfway across the girder. The girder tested was the original design described in the paper.

As to the positions at which strain gauge measurements were taken, he said they had two beams, each consisting of two girders, each consisting of two angles, so that there were eight angles, and readings were taken at nearly every angle. Where the lines on the diagram were dotted, the readings were not very satisfactory for one reason or another, and they were scrapped ; the main readings were taken one at each angle in most cases, but not in all.

He was not sure whether or not he was right in referring to pock marks or whether he should refer to drilled holes. At any rate they had been drilled but were under $\frac{1}{8}$ in. diameter, so that the strain gauge, which had tapered points, fitted much in the same way as did the Whitmore gauge. The reduction in section by this diameter hole was negligible.

Mr. D. A. Cox (Associate-Member), speaking of the girders at the Harlow factory, said that it was essential to allow for the $\frac{3}{8}$-in. expansion under dead and live load and temperature, otherwise secondary stresses would be set up by column restraint. This also meant that wind load must be brought down individually on each column.

No matter how carefully the trusses are made there is difficulty in getting them to exact length. He suggested

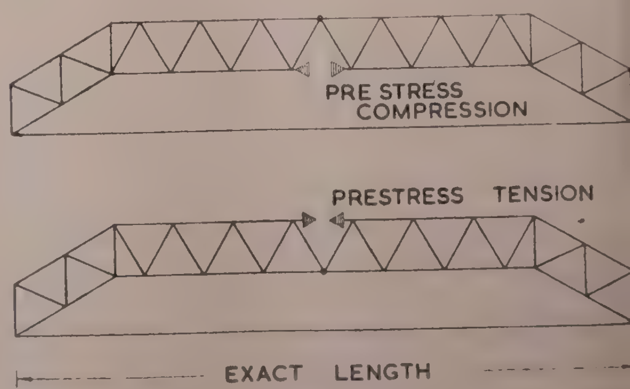


Fig. 20

two methods, either prestressing the bottom chord or top chord of the girder but keeping the tie bars to exact length (see sketch).

When measuring the prestress the strain in the tie bars was measured but it would have been better to measure the strain in the bottom chord where the stresses were much higher.

With regard to the fabrication of girders to exact length, he said that on the second half of the job, not the first half, the beams came out pretty well to within $\frac{1}{8}$ in. of the calculated length, and when prestressed they also came out to within $\frac{1}{8}$ in.

Dealing with the question of expansion of girders under load and change of temperature, one alternative to allowing girders to slide on the top of the columns is to fix the girder to the column. If the girder is welded to the column, say, after prestressing and after dead load has been applied it has been found that the effect on the stresses in the girder is small. On the other hand the base of the columns can be "pinjointed" with the consequent reduction in foundations, etc.

The PRESIDENT, in thanking Mr. Sefton Jenkins for the way in which he had replied to the questions, said the discussion had been a very good one and he hoped Mr. Sefton Jenkins would amplify his remarks later.

(The meeting applaused, and Mr. Sefton Jenkins briefly expressed his appreciation).

Written Discussion

Dr. H. GOTTFELDT (Member) writes: Mr. Jenkins has entrenched himself in a seemingly unassailable position when he says that the order for the work he has described was obtained in open competition. He adds that a comparison of various designs on paper only may be difficult.

While entirely disagreeing with this latter statement—and Mr. Jenkins has himself put forward some very precise sounding claims for his invention of a R.S.J. 24 in. × $7\frac{1}{2}$ in. over a span of 340 ft.—I trust he will at least agree that an exact comparison of the costs of the raw materials is possible.

The structure Mr. Jenkins has described is made of re-rolled steel. What is the price per ton, ex re-rolling mill, of this material, and what would be the corresponding price of new material of the same shape, quality, and quantity? How much of the commercial success of this venture does Mr. Jenkins attribute to the use of this material and how much to the prestressing of it?

My own guess is that 120 per cent. of the success is due to this material and minus 20 per cent. to the prestressing of it—in other words, an ordinary structure made of this material would have been even cheaper than the prestressed one. Mr. Jenkins counters this suggestion by pointing out that an ordinary truss would have had diagonal members of about 12 ft. length, some of them in compression, but I am sure Mr. Jenkins is a sufficiently experienced designer to avoid such long compression members; this could, for instance, be done as shown in Fig. 21. Even if Mr. Jenkins' argument

Fig. 21

had to be accepted it would only be one in favour of a two-pinned frame in accordance with his design, but not in favour of prestressing it.

Mr. Jenkins gives a load of $29\frac{1}{2}$ tons for the test assembly shown in his Fig. 7, which consists of two pairs of twin girders. The load of a single truss is therefore in the region of 8 tons (corresponding to a U.D. load of 25 lb./sq. ft.) and the chord forces of the truss in Fig. 21 would be in the region of 10 tons. In tension this force can be very comfortably sustained by the two angles he uses as chords of his 2 ft. deep truss, and if the $4\frac{1}{2}$ in. × $\frac{1}{4}$ in. plates which he adds to strengthen part of his bottom chord—presumably on account of the compression induced by the prestressing—were used for parts of the top chord in Fig. 21, this, too, could be made from the same two angles. There thus seems to exist a marked disadvantage in having a third chord formed by heavy round bars as prestressing members.

A speaker stated in the discussion that a determinate structure is intrinsically lighter than an indeterminate one. This was perhaps a reference to a paper by Professor Drymael in the December, 1942, issue of the JOURNAL OF THE ROYAL AERONAUTICAL SOCIETY, where the author states: "Corresponding to any redundant structure there is always a statically determinate structure which carries the same loads and which, for the same ultimate stress, is lighter." This confirms that the third chord does, of necessity, increase the total weight. The statement can only be proved for a structure subjected to a single set of loads—a condition that is by and large true for roof trusses—and the proof also neglects the effect of compression. Would Mr. Jenkins agree that prestressing is likely to increase the number of bars that at one time or another are in compression?

In this connection I would like to ask Mr. Jenkins whether there was any reason, other than the need of securing the bottom chords against buckling during prestressing, for arranging the trusses in pairs. If there is none this need has led to an increase in the weight of the purlins: by spacing ordinary single trusses at even intervals the span of the purlins would be nearly halved.

The increase in compressive forces due to prestressing was neatly demonstrated by Mr. Samuely when he showed a slide of the Skylon. The necessary tightening of the guy ropes led to heavy additional forces in the three pylons and the lower part of the Skylon and I am sure Mr. Samuely would gladly have used a method of prestressing these cables which did away with these compression forces if only one could have been found. The Skylon is an example of prestressed cables but not—or if so a not very convincing one—of prestressed steel.

Mr. Jenkins has used ordinary mild steel bars as prestressing members and can therefore claim no advantage from the fact that the cost of high tensile wires does not increase in the same ratio as the strength when compared with mild steel. Anyhow, why waste a great deal of this increased strength by prestressing the wires and producing unwanted compression forces in other parts of the structure? Why not apply our ingenuity to ways and means of utilising this new material, the high-tensile wire, directly as tension member?

There will be difficulties, such as the problem of initially tightening the wires, of connections to other members, possibly of excessive deformations, but they

Fig. 22

do not seem insurmountable and I feel sure that, if they can be overcome, greater benefit will be derived from the direct use of these wires than by first prestressing them. We should aim at self-tightening structures instead of prestressed ones. A common example of this is a suspension bridge and another possibility is shown in Fig. 22. The tie bar of this three-pinned frame

will be sufficiently tight under self-weight and, if it is stressed, under full load, to a stress corresponding to a strain of, say, 0.05, its elongation will be $L/2000$, while the apex will move vertically by $L/^22000H$; neither of these two figures seems excessive for reasonable values of L/H.

In this rough computation I have assumed a stress well below what is customary in prestressed concrete because, as Mr. Jenkins rightly says, in steel structures, whether or not they are prestressed, the force in the members is proportional to the load while the wires in a prestressed concrete structure are little affected by the subsequent loading. But why does he claim this as an advantage, a great advantage, of steel over concrete ? As far as I can see the only result is a lower permissible stress in the prestressing element of a steel structure which must have a factor of safety against unintentional overloading—something that can hardly occur in the wires of prestressed concrete.

The comparison with concrete is abortive in other respects. There the prestressing has two advantages which do not apply to steel structures : the whole of the concrete is brought into action—normally about half of it is useless as far as bending is concerned—and the shear resistance is so greatly improved, even by straight wires, that ties or bent-up bars can be dispensed with. In steel there are not parts without their allotted task and shear is of little consequence.

In conclusion, I would like to avail myself of this opportunity of showing an example of a prestressed structure which I have brought back from a recent visit to Norway—a country that to many may seem far away and of little technical import but where I have found a bewildering variety of structural problems, great vitality in attacking them, and much ingenuity in solving them. I would in particular like to pay tribute to Mr. Ingebrigtsen, until recently Chief Engineer of the Bridge Department of the Norwegian Road Administration, whose untimely death has deprived me and all who have known him of a very esteemed friend.

The example mentioned is shown in Fig. 23. It is a determinate plate girder bridge over five spans. The concrete roadway is anchored to the top flange so as to act in conjunction with it. Prestressing cables are provided in all five spans which will be tightened prior to the placing of the concrete ; the top flanges will thus be in tension and on releasing the prestress the concrete will take most of the compression. These same temporary cables will then be inserted, with opposite curvature, over the supports where they will remain as permanent parts of the structure, inducing tension in the bottom flanges which will here receive compressive stresses from the vertical loads.

It seems hardly possible to get more value out of one and the same cable, but my question regarding the saving in cost was answered immediately and unequivocally : none is expected. The sole reason for this devise is the expected saving in steel, all of which has to be imported from abroad, mainly from Belgium and Germany. Under the economic conditions prevailing in Norway this consideration is of vital importance.

I sincerely hope that Mr. Arild, Mr. Ingebrigtsen's collaborator and successor, will one day find the time to tell us more about the many interesting bridges he has in mind.

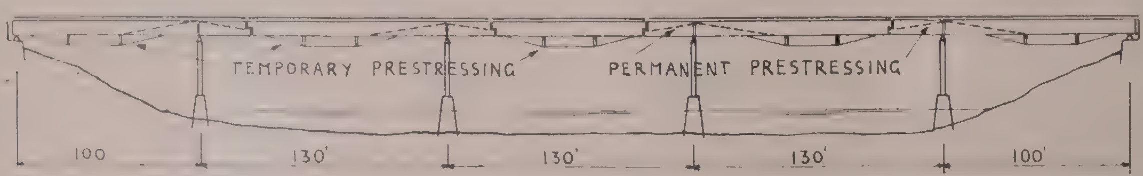


Fig. 23

Mr. SEFTON JENKINS replies : I am grateful to Dr. Gottfeldt for his written contribution as it would appear from it that I have failed to make clear some of the points that I had intended.

Firstly, Dr. Gottfeldt states that he thinks that the sole reason for the success of the work at Harlow is due to the use of re-rolled high tensile material. This is in part true in that the Standard Beam system used produces an extremely light and economical form of construction, not solely, as Dr. Gottfeldt implies, due to the cheaper material (rolled material cost is higher than mild steel, but then so are the stresses used), but mainly because the cost of fabrication of the beams by a semi-automatic process in standard depths considerably reduces the labour costs. In other words, it is essential that if full advantage is to be taken of the manufacturing advantages that accrue from using this High Tensile Beam form of construction that the beam should be of uniform cross-section and construction throughout its length, with certain permissible variations.

Dr. Gottfeldt implies that the design shown in Fig. 21 would be more economical. In weight or in cost ? I doubt whether it would even be lighter in weight, and it would certainly cost more per ton. The total length of the main diagonal and vertical members is over a third more than in the case of Harlow, in addition there is half this amount of " K bracing." The web compression members are some 6 ft. long instead of under 3 ft. Against this must be weighed the weight of the tension member which is relatively small.

As I have mentioned in the discussion in reply to Professor Matheson and Mr. Livesley, the rather theoretical theorem of J. Drymael is all very well when every member in a structure can be made to act to its maximum working stress in pure tension and possibly compression. This means that an R.S.J. or any form of lattice girder where the top and bottom boom are the same throughout their length does not qualify. In other words, with a statically determinate structure one must balance the structural economy requirement of every member being different, against the fabrication requirement that it shall be extremely simple to fabricate with as much as possible of the structure being the same. As an alternative, intelligent use of a statically indeterminate structure can lead to, say, a uniform beam being used at its maximum efficiency at a number of points along its span, especially if it is prestressed so that these points of maximum efficiency can be made to occur where we want them to occur.

Dr. Gottfeldt asks why the girders were put at 24 ft. centres and not at, say, 12 ft. centres. One of the

requirements from a user point of view was that columns should be at not closer centres than 60 ft. $\times$ 24 ft. If girders were put at 12 ft. centres then valley beams would have been necessary, which in turn would have weighed 0.5 lb./sq. ft. Preliminary investigation showed that this would not be saved by halving the centres and would provide a less pleasing appearance. The use of two trusses secured together does counteract the tendency to buckle sideways, but this can be done equally well by fixing secondary beams of the same depth to the sides of the main beam.

A plea is put forward for self-tightening structures such as the three pinned frame in Fig. 22. By fixing the joint at the apex and by overtightening the tie much the same structure is obtained as I have been discussing. The difference lies in that stresses due to positive and negative bending can be more nearly equalised. The structure then acts as a self-tightening structure, so that increased load produces increased stresses acting in opposition to the stresses from the load. In this respect it possesses an advantage over prestressed concrete where the prestress remains virtually constant. This advantage far outweighs the disadvantage due to the use of reduced stresses in the prestressing wires. Needless to say, the shear in the end sloping portion of the girder is reduced, in a way comparable with a curved cable in prestressed concrete, so that the web members of prestressed girder has to take only (in the case of Harlow) a maximum shear of a 40 ft. girder instead of a 60 ft. girder.

Lastly, I should like to thank Dr. Gottfeldt for his interesting example of prestressing steel and concrete by hogging steel girders. This was, I believe, first put forward by Dr. Dischinger, and was briefly discussed by Professor Magnel in his paper to the Institution in November, 1950.

Institution Notices and Proceedings

ELECTION TO HONORARY ASSOCIATESHIP

Professor W. G. Sutton, Principal of the University of the Witwatersrand, has been elected as an Honorary Associate of the Institution in view of his services to the profession of structural engineering.

ORDINARY GENERAL MEETING

An Ordinary General Meeting of the Institution of Structural Engineers was held at 11, Upper Belgrave Street, London, S.W.1, on Thursday, June 24th, 1954, at 5.0 p.m., Lt.-Colonel R. F. Galbraith, M.C., B.Sc., M.I.Struct.E., A.M.I.C.E. (President), in the Chair.

The following members were elected in accordance with the Bye-Laws. Will members kindly note that the elections as tabulated below should be referred to when consulting the Year Book for evidence of membership.

Students

Bebbington, John, of Salford, Lancs.
Bishop, Robert Henry, of Johannesburg, South Africa.
Boddaert, The Jonkheer Edward Patrick, of London.
Chambers, Ronald, of Salford, Lancs.
Chang Weng Onn, of Kuala Lumpur, Malaya.
Cox, William Lawrence, of Salisbury, Southern Rhodesia.
Dickens, William Joseph, of Hednesford, Staffs.
Gibson, Thomas Keith, of Ashton-u-Lyne, Lancs.
Herrod, David Ernest, of Mansfield, Notts.
Houston, Robert Stewart, of Glasgow.
Kler, Devindar Singh, of New Delhi, India.
Low Seng Choy, of Brighton, Sussex.
Marshall, Miss Marline Warwick, of Eldoret, Kenya.
Moore, Ronald Ernest, of Belfast, Northern Ireland.
Porter, John Ellis, of Peterborough, Northants.
Proto, Michael Mathew, of London.
Roddick, John Gordon, of Cardiff.
Singh, Matharu Mohan, of London.
Slack, George Edwin, of Scunthorpe, Lincs.
Wilson, Harold William, of Elland, Yorks.
Wong Yui-Cheong, of London.
Zeegen, Alan Stanley, of London.

Graduates

Adam, Iian Graham, B.Sc.(Eng.) Glasgow, of Kuala Lumpur, Malaya.
Andrew, Alec Albert, of London.
Baird, Jack Alexander, of Erith, Kent.
Baird, John, of Birmingham.
Baker, Stephen, of Darlaston, Staffs.
Beddow, Joseph Derek Harvey, of Woodsetton, Nr. Dudley, Worcs.
Cassidy, Kenneth Evan, of Wembley, Middlesex.
Chapman, Henry Bryan Parry, B.A.(Cantab.), of Ingarsby, Leics.
Collins, Reginald Percy, of London.
Cooper, Bryan Walton, B.Sc.(Civil) Birmingham, D.I.C., of Faversham, Kent.
Corke, Charles Norman, of Liverpool.
Cruse, Ronald James, of London.
Dodsley, Clarence Neville, of Codnor, Derbyshire.
Eachus, Sydney, of Manchester.
Fersztand, Jakob, of London.
Fribbance, William Laurence, of London.
Garrod, Charles John, of Prestbury, Nr. Cheltenham, Glos.
Horwell, David Gordon, of Twickenham, Middlesex.
Hossack, Adam Wilkie, B.Sc.(Civil) Edinburgh, of Galashiels, Scotland.
Hunt, Herbert Arthur, B.Sc.(Tech.) Manchester, of Ripley, Derby.
Iyer, Ramaswami Venkat, B.E. Bombay, of Bombay, India.
Lashmar, Wesley Neville, of London.
Lau Yat Sun, Ph.D., B.Sc.(Eng.) London, D.I.C., of Nottingham.
Lees, Arthur John, of West Bromwich, Staffs.
Ng Peng Khoon, of London.
Preece, Derek Thomas, B.Sc.(Tech.) Manchester, of Ross-on-Wye, Herefordshire.
Quinn, Robert Bruce, B.Sc.(Tech.) Manchester, of Manchester.
Radovic, Igor, of Johannesburg, South Africa.
Rangachari, K., B.E. Madras, of Triplicane, Madras, India.
Ronan, Peter Bryan, of Middleton, Nr. Manchester.
Rutter, Peter Arthur, of New Malden, Surrey.
Saif, Qadir Jabbar, of London.
Sanyal, Kalyan Kumar, B.E. Calcutta, of London.
Schofield, Alfred, of Ashton-u-Lyne, Lancs.
Seager, Michael William, of Chesterfield, Derbyshire.
Souza-Okpofabri, Dazie Sunny, of Lydd, Kent.
Taylor, John, B.Sc.(Eng.) London, of Wigan, Lancs.
Thomson, Andrew Gordon, of Edgware, Middlesex.
Trapp, James Harwood, of Salford, Lancs.
Weerakoon, Chandradasa, of London.
Wilson, William Edward, of London.

ASSOCIATE-MEMBER

JACK, Alexander, of Glasgow.

MEMBERS

BARFOD, Ove Tang, B.Sc. Copenhagen, of Sutton, Surrey.
RENTON, William Langlands, of Edinburgh.
WHITE, Thomas Edward Stephen, B.Sc.(Eng.) London, M.I.C.E., of Wakefield, Yorks.

HONORARY ASSOCIATE

SUTTON, Professor William Godfrey, B.A., B.Sc., M.I.C.E., of Johannesburg, South Africa.

TRANSFERS

Students to Graduates

COX, Anthony James, of London.
HAYMAN, Kenneth Maldwyn, of Denton, Nr. Manchester.
HEWITT, Eric, of Fearnhead, Nr. Warrington, Lancs.
HUGHES, Robert Edward, of Chester, Cheshire.
JONES, William, of Burry Port, Carmarthenshire.
MCNALLY, Eneas, of Manchester.
MUNSCH, Anthony Mark, of Brighton, Sussex.
PULLER, Malcolm John, of London.

Graduates to Associate-Members

AAGAARD, William Valdemar, of Dorridge, Warwickshire.
BURSLEM, John Austin, of Cheadle Hulme, Stockport, Cheshire.
MALLOWS, Dennis Lingwood, B.Sc.(Eng.) Cape Town, of Rondebosch, C.P., South Africa.
MITRA, Asit Kumar, B.E.(Civil) Calcutta, of W. Bengal, India.
RAMA KRISHNA, Hanasoge Suryanarayana Avadhany, B.E. Mysore, of Lonavla, India.
WELLS, Kenneth James, B.Sc.(Civil) Birmingham, of Coventry.

Associate-Members to Members

FOWLER, Richard James, B.Sc.(Eng.), D.I.C., A.M.I. Mech.E., of Birmingham.
MEASURES, Lionel Robert Emery, of Norwich, Norfolk.
WATERS, Thomas Charles, of Culcheth, Nr. Warrington.

OBITUARY

The Council regret to announce the deaths of Professor JOHN ORR (Honorary Associate); STANLEY JOHN BRUFORD, FREDERICK MILTON-COLE (Retired Members); GILBERT JAMES CUBBAGE, JOHN F. CUBBON, D.S.O., M.C., Major WILLIAM RICHARD SIMISTER (Members); ERIC GAYLMER LYTTON-ANDERSON (Associate-Member); JOHN ALEXANDER CALLARD (Graduate).

PROFESSOR JOHN ORR

A correspondent writes :-

Professor Orr was the first professor of Mechanical Engineering at the University of the Witwatersrand, a position he resigned to become Director of the Witwatersrand Technical College.

It was due to Professor Orr's efforts that the chain of Technical Colleges was established along the Witwatersrand. His efforts in promoting the technical education of the engineering apprentices were untiring, and the high standard of education that is to-day set by the Witwatersrand Technical College is in no small measure due to Professor John Orr, the first Director of the Technical College.

RESIGNATION

Notification was given that the Council had accepted with regret the resignation of Peter Baillie (Associate Member).

HONOURS AWARD

In offering their sincere congratulations to the following member on the distinction recently conferred upon him, the Council feel they are also expressing the good wishes of the Institution.

ORDER OF THE BRITISH EMPIRE—O.B.E.
(Military Division)

Lt.-Colonel R. M. Howatt, M.C., T.D. (Member).

PRESIDENTIAL ADDRESS

The Presidential Address for the Session 1954-55 will be given by Dr. S. B. Hamilton, M.Sc., B.Sc.(Eng.), A.R.C.S., M.I.C.E., M.I.Struct.E., on Thursday, October 7th, 1954, at 6 p.m., at 11, Upper Belgrave Street, London, S.W.1.

EXAMINATIONS—JANUARY, 1955

The Examinations of the Institution will next be held at centres in the United Kingdom and Overseas on January 11th and 12th, 1955 (Graduateship), and January 13th and 14th (Associate-Membership).

REPRESENTATION OVERSEAS

The Council have appointed Professor C. A. Hart, T.D., D.Sc., Ph.D., M.I.C.E., F.R.I.C.S. (Associate-Member), as the Institution's Representative in Nigeria.

RESEARCH AWARDS

The Council have instituted a Research Prize Fund, from which awards may be made annually to the author or joint authors of papers describing original research which they have carried out. Research awards may be made for papers read at Headquarters or in the Branches and published in the Journal, or for papers published in the Journal only without being read at an open meeting.

The assessment for such awards will be made annually, but awards will be made only to the contributors of such papers as reach a standard judged by the Literature Committee to be satisfactory.

Work submitted under this scheme must be original and may include any of the following :—

(a) investigations of an experimental or analytical character ;
(b) studies of historical or statistical records ;
(c) improvements in principles or methods of construction ;
(d) research into methods of structural engineering and building, the nature and use of plant and the organisation of engineering work ;
(e) any related or combined studies which are deemed by the Literature Committee to be of a research character.

In cases where the research work described in the paper was not the work of one individual, the names of all the collaborators should be given in the paper.

Awards may take any or all of the following forms :— A research medal ; a diploma ; a money prize.

Application for consideration for a reseach award must be made to the Secretary of the Institution, and in preparing papers for reproduction in the Journal, authors must comply with the conditions laid down for all such contributions. Particulars of these conditions may be obtained from the Secretary.

In judging research papers, the following factors will be considered :—

(a) the nature of the subject and its conclusions ;
(b) the value of the paper in advancing the science and art of structural engineering ;
(c) the standard of preparation and orderly arrangement of the subject-matter.

Research papers will also be eligible for adjudication for the Institution Medals if they comply with the Regulations governing those awards.

The closing date for the receipt of applications in respect of papers published in the Journal between October, 1953, and September, 1954, is October 1st, 1954.

EXAMINATIONS

Preparation for the Examinations of the Institution by Attendance at Technical Colleges

A candidate for Graduateship or Associate-Membership may be able to attend a technical college ; these notes are intended to guide him in choosing the most suitable instruction.

Preparation for the Graduateship Examination

Technical Colleges offer :

(a) Full-time courses for degrees or Higher National Diplomas in Building or Engineering.

(b) Part-time day or evening courses for Higher National Certificates in Building or Engineering.

If he obtains a Higher National Certificate or Diploma complying with Appendix II, Section V, of the Regulations Governing Admission to Membership, the candidate will be exempted from the Graduateship Examination.

Alternatively, he may study subjects selected from the available courses and sit the Graduateship Examination. At technical colleges courses are usually available in Building Science or Engineering Science, Strength of Materials, Theory of Structures and Surveying, but students are not normally allowed to select subjects from National Diploma or Certificate courses unless they can show evidence of sound training in more elementary studies. The advice of the College Authorities should be followed.

Preparation for the Associate-Membership Examination

At some technical colleges there are part-time courses in Structural Engineering which cover the syllabus of the Associate-Membership Examination. At other colleges the candidate must rely on Higher National Certificate courses or on advanced courses in Building, Civil Engineering or Municipal Engineering ; these cover only part of the requirements for the Associate-Membership Examination.

Colleges in List "A" provide at least two years of instruction in Theory of Structures and in Structural Engineering Design and Drawing up to Associate-Membership standard. They also give instruction in Structural Specifications, Quantities and Estimates.

List "A"

- Bath Technical College.
- Belfast College of Technology.
- Birmingham College of Technology.
- Bolton Municipal Technical College.
- Bradford Technical College.
- Bridgend Technical College.
- Chesterfield College of Technology.
- Derby Technical College.
- Dudley and Staffordshire Technical College.
- Glasgow Royal Technical College.
- City of Liverpool College of Building.
- L.C.C. Brixton School of Building, S.W.4.
- L.C.C. Hammersmith School of Building and Arts and Crafts, W.12.
- Manchester College of Technology.
- Middlesbrough, Constantine Technical College.
- Nottingham and District Technical College.
- Salford, Royal Technical College.
- South-East London Technical College, Lewisham Way, S.E.4.
- South-West Essex Technical College, Walthamstow, E.17.
- Stafford, County Technical College.
- Stockport College for Further Education.
- Twickenham Technical College.
- Willesden Technical College, N.W.10.

Colleges in List "B" provide instruction in Theory of Structures from which the student may reach Associate-Membership standard, but instruction in Structural Engineering Design and Drawing and in Structural Specifications, Quantities and Estimates is not usually so complete.

List "B"

- Brighton Technical College.
- Cardiff Technical College.
- Edinburgh, Heriot-Watt College.
- Huddersfield Technical College.
- Leeds College of Technology.
- London, Battersea Polytechnic, S.W.11.
- London, Northampton Polytechnic, E.C.1.
- L.C.C. Westminster Technical College, S.W.1.
- Newcastle-upon-Tyne, Rutherford College of Technology.
- Plymouth and Devonport Technical College.
- Preston, Harris Institute.
- Rotherham College of Technology.
- Wigan Mining and Technical College.
- Woolwich Polytechnic, S.E.19.

Students are advised to take the organised courses in Structural Engineering where these are available.

LONDON GRADUATES' AND STUDENTS' SECTION

A visit to the Atomic Research Establishment at Harwell, has been arranged for Saturday, September 18th. The number of visitors is limited to 24, of British nationality. Transport by coach from the Institution has been arranged and details will be sent to those who wish to participate in the visit, on application to the Hon. Secretary.

The next indoor meeting of the Section will be held at 11, Upper Belgrave Street, London, S.W.1, on Tuesday, October 19th, at 6 p.m., when Mr. L. Scott White, O.B.E. (Past President), will read a paper on "Government Offices, Whitehall Gardens, the special problem of the Re-siting of an Historic Building." This will be followed on Saturday, October 23rd, by a visit at 10 a.m., to the site of the work to inspect the Historic Building, together with the new construction now in progress.

Tuesday, November 23rd. A meeting of the Section will be held at 11, Upper Belgrave Street, London, S.W.1, at which members of the Section are invited to give short talks of 15-20 minutes' duration on various aspects of Structural Engineering. A prize will be awarded for the most meritorious contribution. Will those wishing to contribute please advise the Hon. Secretary ?

Hon. Secretary : J. A. Pope, 53, Cranleigh Drive, Leigh-on-Sea, Essex.

BRANCH NOTICES

Lancashire and Cheshire Branch

The Inaugural Meeting of the Session will be held at the Walker Art Gallery, Liverpool, on Friday, October 15th, at 7.15 p.m., when the Chairman's Address will be given by Mr. W. D. Blades, M.I.Struct.E.

Following the Address the following film will be shown: "Soils and Foundations—Loch Sloy."

The President and the Secretary of the Institution will attend the meeting.

Joint Hon. Secretaries : A. S. Sinclair, A.M.I.Struct.E., 17, The Circuit, Cheadle Hulme, Cheshire ; M. D. Woods, A.M.I.Struct.E., 58, Spring Gardens, Salford.

MIDLAND COUNTIES BRANCH

The Annual Dinner of the Branch will be held at the Botanical Gardens, Birmingham, on Saturday, October 9th.

The opening meeting of the Session will be held on Friday, October 22nd, at the James Watt Memorial Institute, Birmingham, at 6 p.m., when the Chairman's Address will be given by Mr. W. Phillips, M.Eng.(Sheffield), M.I.C.E., M.I.Struct.E.

The President and the Secretary of the Institution will be present on both occasions.

Hon. Secretary : L. A. Firminger, A.M.I.Struct.E., 656, Chester Road, Erdington, Birmingham, 23.

GRADUATES' AND STUDENTS' SECTION

A Joint Meeting will be held with the Graduates' and Students' Section of The Institution of Civil Engineers, at the Birmingham Civic Centre, on Friday, October 29th, at 6 p.m., when a paper will be given on "Site Investigation for Foundations," by R. D. Mackey, B.Sc.(Hons.), A.M.I.C.E. (Graduate).

Hon. Secretary : H. L. Bramwell, 139, Wood Lane, Handsworth Wood, Birmingham, 20.

NORTHERN COUNTIES BRANCH

The opening meeting of the Session will be held at Middlesbrough, on Tuesday, October 12th, and will be attended by the President and the Secretary of the Institution.

Hon. Secretary: Capt. O. Lithgow, A.M.I.Struct.E., 4, Stoneleigh Avenue, Acklam, Middlesbrough.

NORTHERN IRELAND BRANCH

The opening meeting of the Session will be held at the College of Technology, Belfast, on Tuesday, October 5th, at 6.45 p.m., when Mr. J. Singleton-Green, M.Sc., M.I.C.E., A.M.I.Mech.E. (Member of Council), will give a paper on "Concrete as an Engineering Material."

The meeting will be preceded by tea at the Overseas League Premises, Wellington Place, Belfast, at 6 p.m.

Hon. Secretary : A. H. K. Roberts, B.A., B.A.I., M.I.Struct.E., M.I.C.E., "Barbizon," 26, Dunlambert Park, Belfast.

SCOTTISH BRANCH

The opening meeting of the Session will be held on Monday, October 25th, and the Branch Annual Dinner and Dance will be held at Glasgow, on Tuesday, October 26th.

The President and the Secretary of the Institution will attend on both occasions.

SOUTH-WESTERN COUNTIES BRANCH

The opening meeting of the Session will be held on Saturday, November 13th, at the Duke of Cornwall Hotel, Plymouth, at 6.30 p.m., when the Chairman's Address will be given by Colonel R. Hazzledine, O.B.E., T.D., M.I.Struct.E.

The President and the Secretary of the Institution will attend the meeting, which will be followed by an Informal Dinner.

Joint Hon. Secretaries : E. W. Howells, M.I.Struct.E., 10-12, Market Street, Torquay ; C. J. Woodrow, 23, Torland Road, Hartley, Plymouth.

WALES AND MONMOUTHSHIRE BRANCH

The opening meeting of the Session will be held at the South Wales Institute of Engineers, Cardiff, at 6.30 p.m., on Tuesday, October 19th, when the Chairman's Address will be given by Mr. V. M. Bell, M.Eng., M.I.C.E., M.I.Struct.E.

The President and the Secretary of the Institution will attend.

Hon. Secretary : K. J. Stewart, A.M.I.C.E., A.M.I. Struct.E., 15, Glanmor Road, Swansea.

WESTERN COUNTIES BRANCH

The opening meeting of the Session will be held in the University of Bristol Geology Lecture Theatre (entrance University Road), on Wednesday, October 20th, at 6 p.m., when the Chairman's Address will be given by Mr. E. N. Underwood, B.Sc.(Eng.), M.I.C.E., M.I.Struct.E.

The President and the Secretary of the Institution will attend the meeting, which will be preceded by tea at 5.30 p.m.

Hon. Secretary : E. Hughes, A.M.I.Struct.E., 23, Southdown Road, Westbury-on-Trym, Bristol, 9.

YORKSHIRE BRANCH

The opening meeting of the Session will be held at Leeds on Thursday, October 13th, and will be attended by the President and the Secretary of the Institution. The Chairman's Address will be given by Mr. Leslie Preston, M.I.Struct.E.

Hon. Secretary : E. Wrigley, A.M.I.Struct.E., 17, The Drive, Alwoodley, Leeds.

UNION OF SOUTH AFRICA BRANCH

Branch Hon. Secretary : A. E. Tait, B.Sc., A.M.I.C.E., A.M.I.Struct.E., P.O. Box 3306, Johannesburg. During week-days Mr. Tait can be contacted in the City Engineer's Department, Town Hall, Johannesburg. 'Phone : 34-1111. Ext. 257.

Natal Section, Hon. Secretary : E. G. Bennett, A.M.I.Struct.E., c/o Reinforcing Steel Co., Ltd., P.O. Box 49, Merebank, Durban.

Cape Section, Hon. Secretary : R. Stubbs, M.I.Struct.E. P.O. Box 1692, Cape Town.

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The President, 1954-1955

Dr. S. B. Hamilton, whom the Institution welcomes as President for the Session 1954-55, brings to his office a wide and varied experience in several spheres of structural engineering—in research, in education, and in the practical work of everyday engineering.

Stanley Baines Hamilton was born at Lowestoft. He was educated at Halifax Technical College, where he won distinction, being awarded the Schofield Gold Medal for the most successful student in the Mechanical Engineering Department for the Session 1907-1908, and gaining a Royal Exhibition. He went up to London University in 1908 and studied at the Royal College of Science. In 1910, he qualified for Associateship of the Royal College of Science, took his degree of B.Sc.(Engineering) and was awarded a Daimler Major Scholarship. He then underwent training for two years as a pupil with the Daimler Company, Ltd., at Coventry, and in 1913 joined the Great Northern Railway as a pupil under Charles T. Brown, the Chief Engineer.

Dr. S. B. Hamilton, M.Sc., B.Sc.(Eng.), M.I.C.E., M.I.Struct.E.

Dr. Hamilton took up his first appointment in 1914 with Highways Construction Ltd., as Supervisor on Bitumastic Road Surfacing Contracts. His career was interrupted by the first World War, and from 1914 until 1919 he was engaged in military service. He served first with the Royal Garrison Artillery at Dover and in Singapore, where he was transferred to the Royal Engineers and served as Division Officer at Blakang Mati, Singapore, and afterwards at Enniskillen, Northern Ireland. In Singapore, he was chiefly engaged on land drainage works for the prevention of malaria, besides maintenance of Army buildings and installations.

On his return to civilian life in 1920, Dr. Hamilton was appointed Assistant Engineer to R. Young & Co., Civil Engineers and Contractors, at Penang, Straits Settlement, where he remained for nearly four years. His work there included estimates, designs and supervision of constructional work, mainly in reinforced concrete, for buildings, jetties, sea walls and watergates.

In 1924, he joined the Chief Structural Engineer's Division of H.M. Office of Works, where his work included the design of earthquake resisting buildings for Japan and the structural survey of the Central Telegraph Office and of parts of the British Museum. Dr. Hamilton remained with the Office of Works until 1936, when he went to the Designs Branch of the War Office. In 1943, he took up an appointment at the Building Research Station of the Department of Scientific and Industrial Research, which he holds at present. Here, he has been concerned with the preparation of leaflets on the Repair of Damaged Buildings, the examination of schemes for non-traditional houses and contributions to various publications.

During the course of his career, Dr. Hamilton has acquired the following additional academic qualifications : University of London Teachers' Diploma, 1929 ; Master of Science Degree (London), in History, Methods and Principles of Science, 1933, and Doctor of Philosophy (London), in History and Philosophy of Science, 1950.

Dr. Hamilton was President of the Newcomen Society for the Study of the History of Engineering and Technology in 1944-1946, and has published various papers in the Transactions, on historical subjects. He is also a Member of the Institution of Civil Engineers.

Dr. Hamilton has had considerable teaching experience, at the Westminster Technical Institute from 1924 until 1939, and at Brixton School of Building between 1939 and 1944 ; his main subjects were Theory of Structures (Advanced) and Strength of Materials.

The President's Association with the Institution of Structural Engineers dates from 1924, when he was elected a Member. Since then, he has taken an active interest in all the branches of its work and has served on the Standing Committees dealing with the following matters : Finance and General Purposes, Membership, Literature, Science, Education and Examinations, Legislation, Representation and Bye-Laws. He has been Chairman of the Finance and General Purposes Committee, Membership Committee and Education and Examinations Committee. and also of the Masonry Sectional Committee. Dr. Hamilton was elected to the Council for the first time in 1938 ; he has held the offices of Honorary Secretary (1947-48), Honorary Treasurer (1948-49) and Vice-President (1949-54).

He has rendered invaluable service to the Institution by representing it on such bodies as the Engineering Joint Examination Board, of which he is Chairman ; the Ministry of Education Joint Committee on Higher National Certificates in Civil Engineering and the Associated Examination Board for the General Certificate of Education ; the Ministry of Labour and National Service (Appointments Department, Technical and Scientific Register) Civil Engineering Advisory Committee ; the City and Guilds of London Institute (Department of Technology) Advisory Committee on Structural

(*Continued on page* 283)

Foundations, Underpinning and Structural Problems at the Daily News Building in the City of London*

By Frederick W. Slatter, M.I.Struct.E., M.Cons.E., M.Inst.W., M.Soc.C.E.(France) and Arthur Brown, A.M.I.Mech.E., M.Soc.C.E.(France)

Synopsis

The subject largely deals with the constructional work below ground level for a heavy-type structure that has recently been erected in the City of London. Like so many buildings in this area, the site is extremely confined and is hemmed in on two of its flanks by property of a similar nature. The text is mainly devoted to the practical aspects of some of the more interesting foundation and underpinning problems that are peculiar to buildings in the densely built-up areas. In such projects working space is very limited and, due to this, all design considerations are inseparable from the constructional aspect.

When planning the rebuilding of premises of this type much of the foundation work, where it abuts adjacent property, is often speculative and it is not until excavation and inspection holes are open that positive proposals can be made. With this thought in mind, it will be appreciated that there must be a continual flexibility in design consideration that will meet the many varying conditions as they arise. Many of the original designs for founding the external stanchions; that were based on information obtained from old record drawings of the adjacent premises had to be completely revised. Excavation work proved these latter drawings to be incorrect and the final decisions were often largely controlled by practical considerations. The authors have endeavoured to concentrate on describing some of the more interesting foundation units employed in the above premises, also their construction, and have, at the same time, tried to show the close relationship between design and practical considerations.

Introduction

The building recently completed replaces an earlier structure destroyed by enemy action during May, 1941. Like its predecessor, the new premises are a complete "printing house," embodying all the special features of construction and layout that are vital to ensure the uninterrupted flow of newsprint for a group of the national daily newspapers.

The original structure of the building was typical of so many of its day, i.e., part steel framework and load-bearing walls with brick vaulted retaining walls to the basement. In the new building there is a complete steel-framed superstructure with solid reinforced concrete [illegible] nd additional deep basements also constructed [illegible] orced concrete.

General Description of Premises

[illegible] ll property in the City of London, the site on [illegible] new building is constructed is extremely [illegible] and the commercial value of the ground is [illegible] high, with the consequence that the whole of the site had to be fully developed. The site is bounded on the north and south flanks by existing buildings, on the east face by a narrow alley and the front elevation, which faces west, abuts a typical London side-street carrying a regular heavy volume of vehicular traffic.

Fig. 1 shows a general plan of the building and a layout of the foundations, whilst Figs. 2 and 3 show typical cross-sections from which it will be seen that the total area occupied is by no means large. The structure comprises seven floors, including a basement and sub-basement, and the main roof level, excluding pent-houses, rises to a height of 84 ft. above pavement level. There are also many intermediate floor levels and all are served by two main staircases and three lifts.

Many special rooms are included to accommodate printing and allied plant, the whole scheme being carefully designed to produce the maximum efficiency and split-second timing that is so vital to the newspaper industry. To meet bye-law conditions of light and air, it was necessary to set back the building storey by storey on its north, east and south faces at third floor level and introduce mansard slopes with dormer windows from this point to roof level.

Site and Boundaries

To obtain the necessary floor space for the newly-planned plant and production, the present building is provided with an additional sub-basement over part of the site which did not appear in the earlier structure. The floor level of this averages 27 ft. below pavement level, whilst the general basement level is approximately 16 ft. below pavement level. Referring again to Fig. 1, it will be seen that the north boundary is immediately flanked by printing premises belonging to the NEWS OF THE WORLD. This latter building is a relatively modern structure—completely steel-framed except for its south party wall, which is free standing, and is adjacent to the DAILY NEWS building. This wall is formed in three parts, i.e., the western third being some 100 ft. high, the centre third 35 ft. high with a lighting well over and the eastern third again 100 ft. high. In order to construct the sub-basement, the western (front) section of this wall was underpinned to a depth of 8 ft., a detailed description of which follows later in the text.

The south flank is bounded on its eastern half by premises belonging to the Associated Newspapers, Ltd., having a sub-basement approximately 50 ft. deep and the remaining half, by existing DAILY NEWS property, with a sub-basement 37 ft. deep. In addition, on the south side is a public thoroughfare—Magpie Alley—running partly through the DAILY NEWS property and partly over Associated Newspapers premises.

Foundations

Generally, the practical aspect of these is complex and the difficulties in construction were increased by the

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[illegible] read before the I[illegible] of Structural Engineers at 11, Upper Belgrave Street, Lon[illegible] W.1, on Thursday, October 28th [illegible], at 6 p.m.

confined nature of the site. The major portion of the ground on which the various foundation units were cast was blue clay and the safe ground pressure adopted for design was 3 tons per square foot. In the isolated 45 ft. deep section of the building in the south-west corner which was extremely confined, the safe ground pressure at this level was taken at 4 tons per square foot. Due to the relatively high incidence of loading approximately 80 per cent. of the available site was loaded to capacity.

"News of the World" Party Wall

This wall had been partly underpinned on two previous occasions, initially when the basement in the earlier DAILY NEWS building was taken lower than the original NEWS OF THE WORLD building and, subsequently, when the present NEWS OF THE WORLD premises were constructed.

In this present structure the sub-basement foundations are situated approximately 8 ft. lower than those existing. As previously mentioned, the party wall is free standing and non-load bearing but the steel framework to the adjacent property is supported on R.S.J. spreader beams bearing on the earlier underpinning.

After completion of all demolition, a general illustration of which is shown in Fig. 3a, the first construction work executed was the underpinning to the western section of the above wall. In areas remote from the main raft foundations, this was carried out in alternate 5 ft. widths. This underpinning was to extend only as far as the NEWS OF THE WORLD face of the party wall in order to make it convenient for the adjoining basement to be deepened at some future date without the new foundation encroaching inside the NEWS OF THE WORLD building. (It is pointed out that it was necessary to trim previous underpinning during the present construction.)

This unorthodox method necessitated careful preparation and the following procedure was adopted.

Each section was excavated in 3 ft. deep units at the back of which a 6 inches-thick *in situ* reinforced concrete skin wall was constructed and this concrete membrane served as a permanent poling board. The shuttering to this concrete had specially prepared walings, and while the concrete was still green, these were strutted back to the excavated face by timber struts fitted with hydraulic jacks. On reaching the required predetermined pressure, the timber struts were then tightened up by means of folding wedges, thereby releasing the load from the jacks. As the wedges were driven home, the load sustained by the jacks was gradually released and when the load was reduced to zero the whole of the pressure was taken by the struts leaving the jacks free for the next operation. This procedure ensured a positive support to the excavated face beneath the existing foundation and at the same time firmly embedded the new concrete "poling board" into the clay, thus reducing the possibility of any voids remaining between the concrete and clay faces.

Having formed the first section of permanent shuttering, succeeding approx. 3 ft.-deep sections were constructed in the same manner until the required depth, i.e., 8 ft., was reached. At this stage the underside of the existing foundation was thoroughly cleaned off,

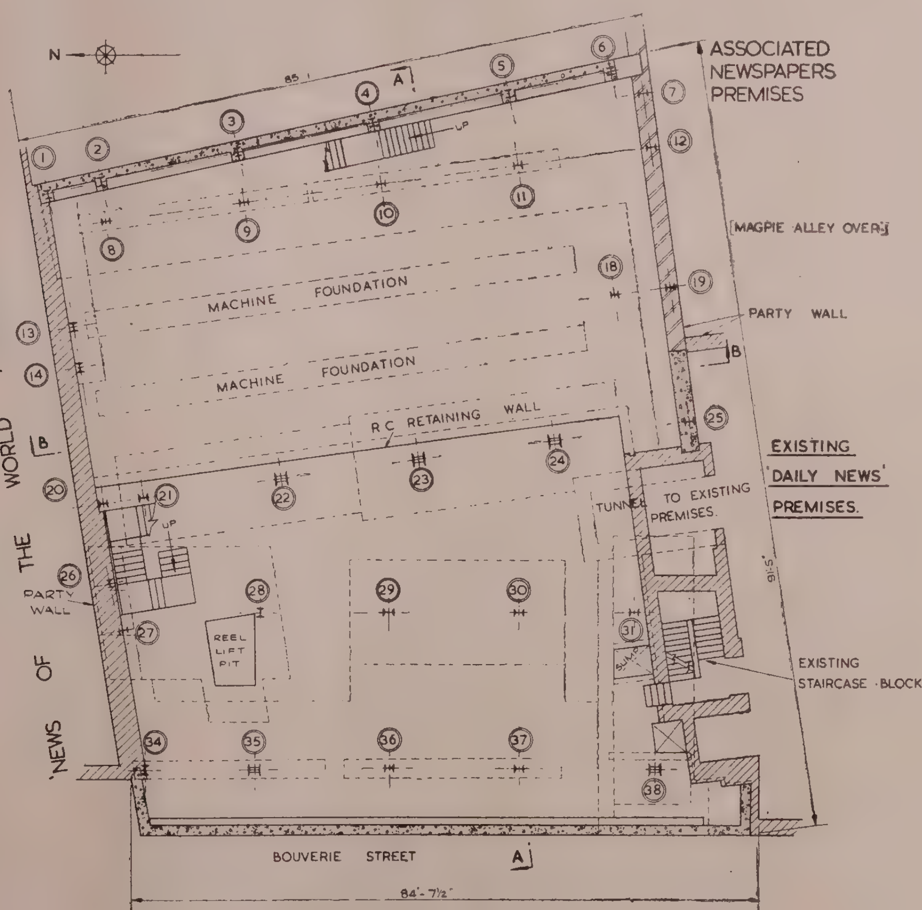


Fig. 1

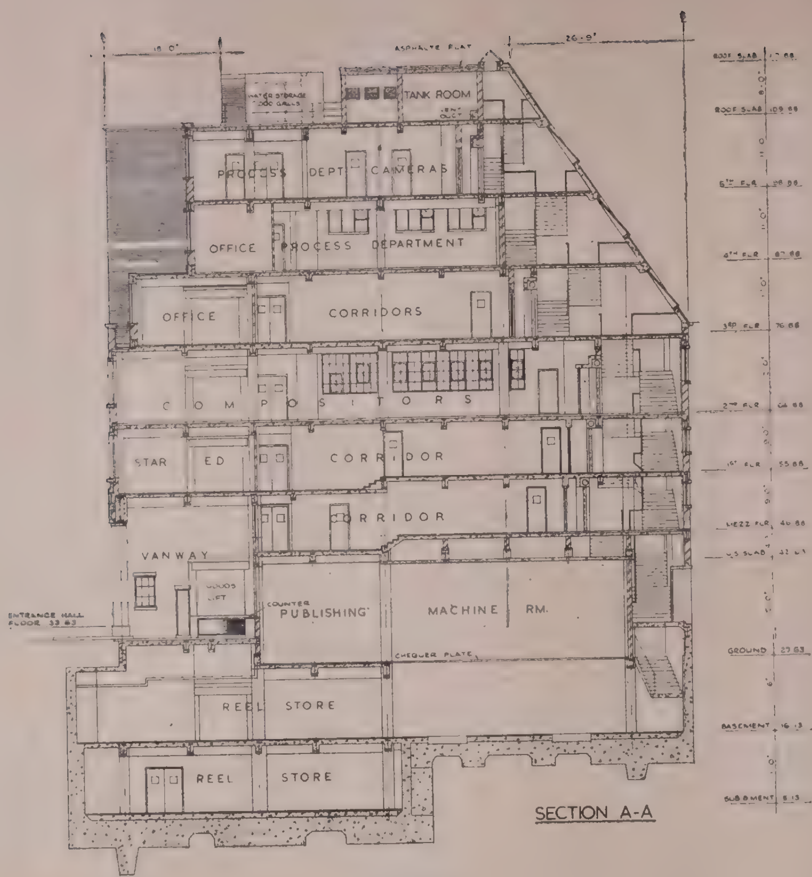
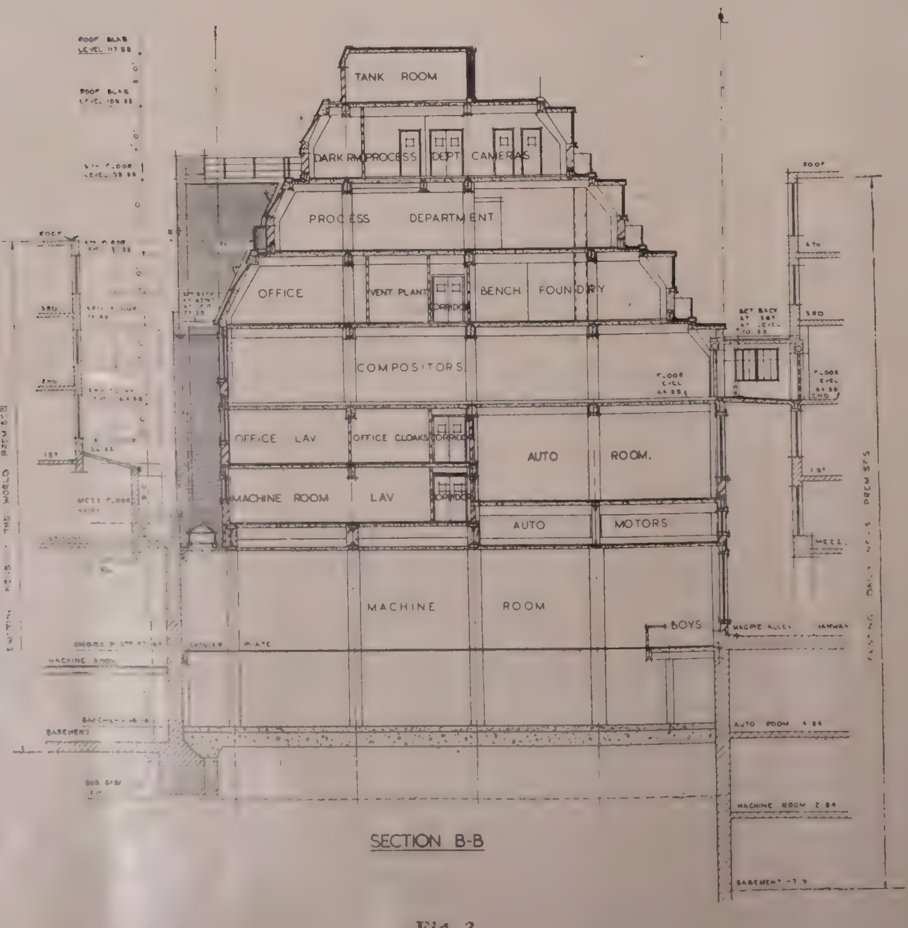

TANK ROOM
PROCESS DEPT CAMERAS
OFFICE PROCESS DEPARTMENT
OFFICE CORRIDORS
COMPOSITORS
STAR ED CORRIDOR
CORRIDOR
VANWAY
PUBLISHING MACHINE RM.
REEL STORE
REEL STORE
SECTION A-A

Fig. 2


TANK ROOM
DARK RM PROCESS DEPT CAMERAS
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OFFICE VENT PLANT BENCH FOUNDRY
COMPOSITORS
OFFICE LAV OFFICE CLOAKS AUTO ROOM.
MACHINE ROOM LAV
AUTO MOTORS
MACHINE ROOM
BOYS
SECTION B-B

Fig. 3

after which the void formed was filled with Quality IIIA concrete (L.C.C.), leaving a 3-inches space at the top. This aperture between the new and existing concrete surfaces was finally filled by ramming in half dry 1 : 1 sand and cement to complete the underpinning operation. A typical detail of this work is shown in Fig. 4.

Retaining Walls

Whilst these operations were being carried out, work proceeded on the excavation for the Ashentree Court (east side) and Bouverie Street (west side) retaining walls.

The digging on the Bouverie Street frontage was to be some 30 ft. below pavement level and, since the road over is traversed by very heavily loaded lorries bringing reels of paper to the various printing houses, great care was necessary in the construction of this wall. To ensure continuous stability to the Bouverie Street face, it was decided to construct the northern and southern corners of this side of the basement before excavating the centre section.

In view of the considerable depth of the sub-basement, due consideration was given to the problem of ensuring that the area would be waterproof. Borehole tests carried out before the commencement of the contract did not divulge any standing water at the proposed excavated depths and it was, therefore, assumed that the water problem would not be very serious. However, a small but continuous flow of water did appear in the Bouverie Street retaining wall excavation emanating from the north face. To resist this, the specification for the concrete included a waterproofing admixture and poker type vibrators were used in all placing. All vertical and horizontal joints were formed with inverted dovetail sections. On completion of all daywork joints, laitence was washed from the top of the freshly-placed concrete with a high pressure water spray—this was carried out within $1\frac{1}{2}$-2 hours after placing. Surfaces treated in this manner produced an extremely clean appearance with the aggregate exposed and subsequently this was coated with a cement grout immediately prior to the next casting. The joint thus formed was free of foreign matter and the quality of its water resistant properties proved extremely successful.

Both the aforementioned walls were basically designed as cantilevers, and in each case the toe to the wall included a continuous beam foundation which supported some of the main stanchions.

Referring to the Ashentree Court retaining wall, this differs from the Bouverie Street wall in so far that the stanchions on this external face are supported directly on the wall at ground floor level. This procedure was adopted so that the whole of the basement could be constructed, thus leaving a free unencumbered area in which the steel erectors could ultimately work. The alternative to this method was to found the stanchions on this face at basement level which would have involved erecting the lower lengths of these members prior to the casting of the wall. In view of the complexity of this operation also the difficulty of lining the stanchion shafts, the former method was adopted. To distribute the stanchion loads, the wall was thickened beyond that required for a pure cantilever unit and pockets were formed just below ground floor level to receive the stanchion bases.

In the course of excavation, precast concrete poling boards were adopted as a support to the earth face behind the timber walings and these formed permanent shuttering to the outside face of the wall. The concrete poling boards were 3 ft. × 1 ft × 2 in. thick, and were provided with two $1\frac{1}{2}$-in. diameter holes through the 2-in. thickness for the purpose of forcing grout between the board and the clay face. Unlike timber poling boards which would have to be withdrawn after concreting and the remaining void filled, grout was forced through the holes, thus enabling all voids between the concrete and the cut clay face to be filled solid, thus preventing any local settlement in the road.

The problem of designing the strutting to the excavated face to facilitate the placing of reinforcement and arrange a practical concreting programme was carefully considered at the outset. In deciding the complete arrangement no factor could be settled without full consideration of all allied problems. Shown in

Fig. 3a (*Photograph by courtesy of* DAILY NEWS LTD.)

Fig. 5 are typical details of the procedure adopted on the Bouverie Street wall. The concrete poling boards were held back to the clay face in the first instance by a timber sub-waling and this in turn was retained by a series of timber spacing blocks, spaced conveniently to allow the reinforcement to be fixed. These blocks were restrained by the main walings which developed their support from the struts bearing on the dumpling. Each frame of timber erected as the excavation proceeded was 3 ft. below the preceding frame, the poling board length was made the same to simplify the support to this latter unit. Fig. 5a shows the nearly completed Bouverie Street wall.

At the south-west corner of this site, a problem existed where two of the stanchions in the new building (Nos. 38

from each unit so that it could be lapped with each succeeding unit. As the openings formed at each operation were made larger than is normal for conventional underpinning, temporary steel struts bearing on concrete pads were inserted beneath the existing foundation and these were eventually cast in with the new raft/toe. Fig. 6 shows a general arrangement of the foundation. This section of the work, as a separate unit, required seven months to complete. Some idea of the excavation for this operation is shown in Fig. 6a, and Fig. 6b is an illustration of the shaft after removal of the timbering.

Another interesting combined foundation and underpinning operation was carried out on the north side of the building, adjacent and underneath the western section of the NEWS OF THE WORLD party wall. Behind

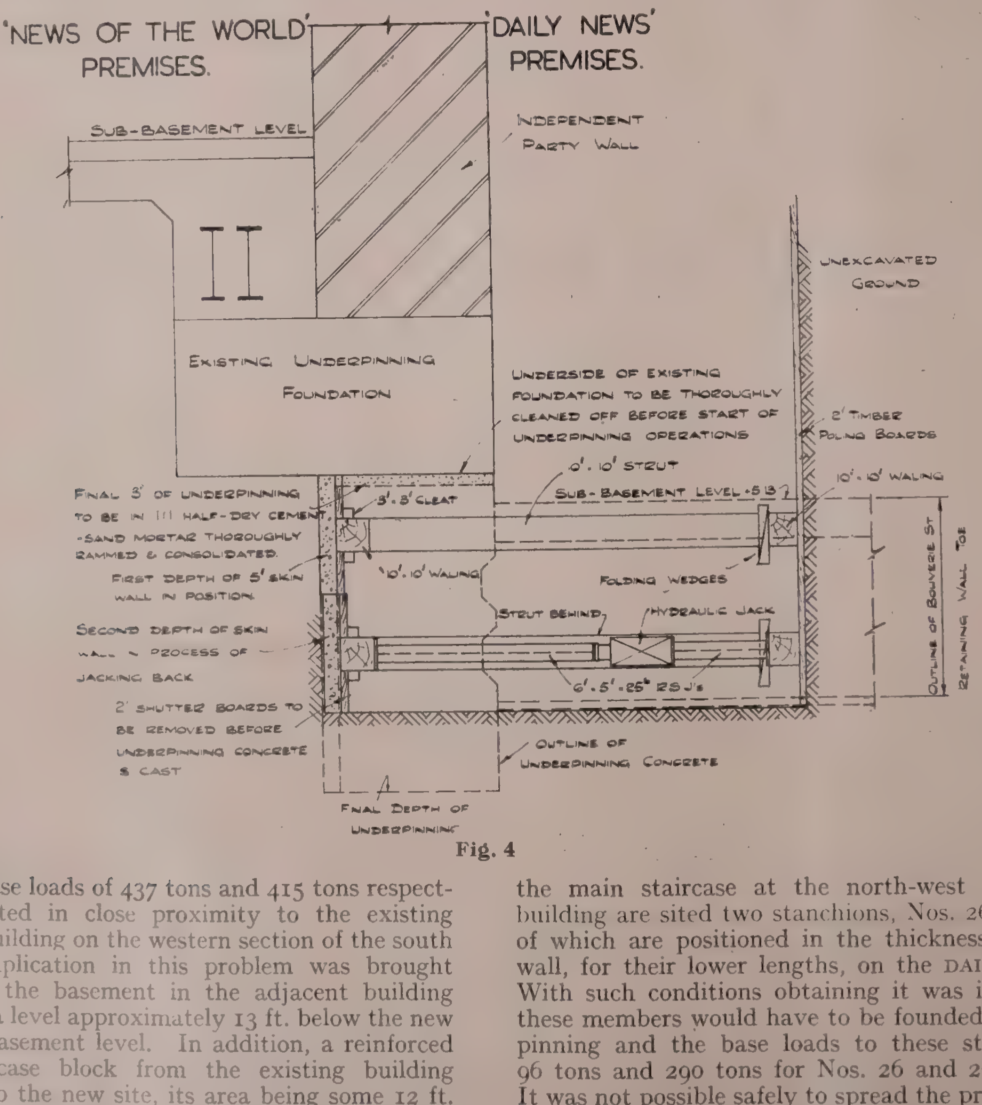


Fig. 4

and 31 with base loads of 437 tons and 415 tons respectively) were sited in close proximity to the existing DAILY NEWS building on the western section of the south flank. A complication in this problem was brought about due to the basement in the adjacent building being sited at a level approximately 13 ft. below the new building sub-basement level. In addition, a reinforced concrete staircase block from the existing building projected on to the new site, its area being some 12 ft. wide × 37 ft. long. This appendage was to be preserved for future access between the new and old buildings at ground level and had to be maintained during the construction of the new building since it also housed [illegible]tch rooms and ink storage tanks. To provide adequate foundation for the two stanchions, it was decided to form a combined base to be constructed at [illegible]l and partly below the aforementioned staircase block. This particular foundation also formed the toe to the Bouverie Street retaining wall at the south end.

The maximum depth of underpinning for this was 8 ft. deep × 6 ft. high and the overall length 30 ft. Construction was carried out in five units of varying lengths, the reinforcement being arranged to project

the main staircase at the north-west corner of the building are sited two stanchions, Nos. 26 and 27, both of which are positioned in the thickness of the party wall, for their lower lengths, on the DAILY NEWS side. With such conditions obtaining it was inevitable that these members would have to be founded on the underpinning and the base loads to these stanchions were 96 tons and 290 tons for Nos. 26 and 27 respectively. It was not possible safely to spread the pressure through the underpinning concrete and it was decided to form a combined foundation with stanchion No. 28 (load 548 tons), which was sited 19 ft. 6 in. directly to the south of the stanchions previously mentioned, approximately on a centre line between the two. Having decided that a raft was a practical solution to support the three stanchions, the method of constructing it was carefully considered. The most important feature of the design was the complete continuity of the raft with the underpinning, and to effect this the following procedure was adopted.

The size of the raft needed to be 18 ft. wide × 25 ft. long and the 18 ft. width had to be constructed partly under the party wall. It was considered safe to carry

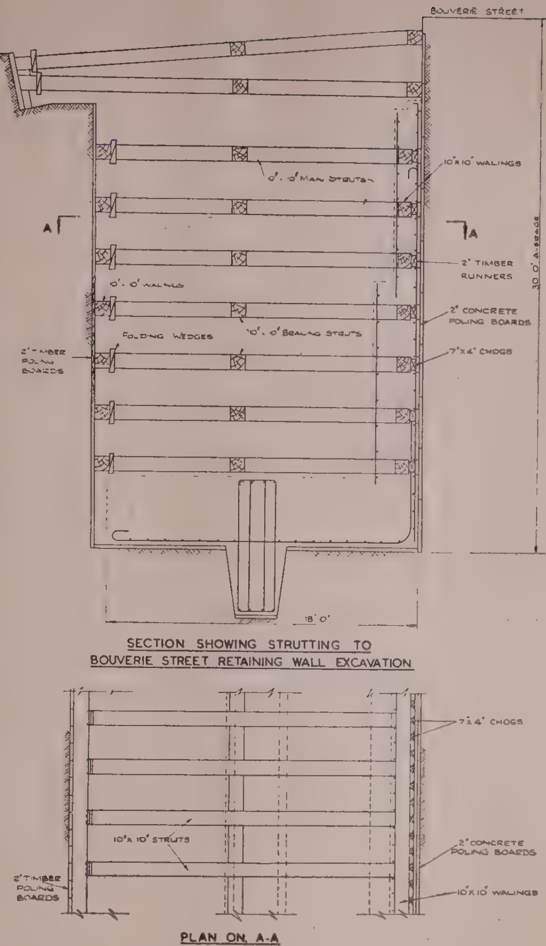


Fig. 5

out underpinning in widths up to a maximum of 6 ft. at one time, therefore, this raft at its northern end was made up of three 6 ft. strips. Each section of underpinning constructed was cast complete with the necessary raft reinforcement, a portion of which was left projecting to be spliced with steel subsequently fixed in the remaining raft section. Instead of the final pinning up being executed as described before, the top side of the concrete was kept down 3 ft. 9 in. below the existing foundations and pinned up at the first stage in engineering brickwork laid with dry joints. The express reason for this was because the brickwork had to be taken out again at a later stage to leave room for the insertion of a grillage.

Stanchions 26 and 27 were 6 ft. 8 in. apart (east to west) and were to be supported on a foundation cast in three separate widths for part of its length with no lateral connection. To ensure the whole foundation would work as a monolithic unit, a combined grillage formed the base to these stanchions, thereby spreading the load across the foundation. In the temporary blue brick underpinning mentioned above, three 2 ft.-wide pockets were left forming openings between the underside of the existing foundation and the new underpinning concrete, these openings extending back to the concrete skin wall.

After this work was completed, the excavation for the main section of the raft connected to this underpinning proceeded. As will be seen in Fig. 1, the construction was further complicated by the presence of a lift pit adjacent to stanchion 28. Since there was little room between this stanchion and the southern edge of the lift pit, it was not convenient to adopt a bloom base to distribute the load to the foundation raft. In view of this circumstance plated spreader beams were chosen as a suitable medium to transfer the load from the stanchion to the raft, an arrangement that suited the physical conditions and at the same time evenly distributed the pressure across the 18 ft.-wide foundation at

Fig. 5a

its southern end. Reference to Fig. 7 will indicate the construction of this foundation where it forms part of the underpinning and Fig. 7a shows the temporary underpinning completed with the main raft reinforcement ready for concreting.

After the completion of the main raft construction, steel needles attached to R.S.J. posts were inserted into the pockets left in the engineering brickwork, their outer ends being supported on concrete posts 3 ft. 9 in. clear of the party wall. The existing foundation was then pinned up in 1 : 1 half dry sand and cement on the steel needles, after which all the brickwork was taken out. This left a clear opening underneath the existing foundation, approximately 18 ft. long and 3 ft. high, being

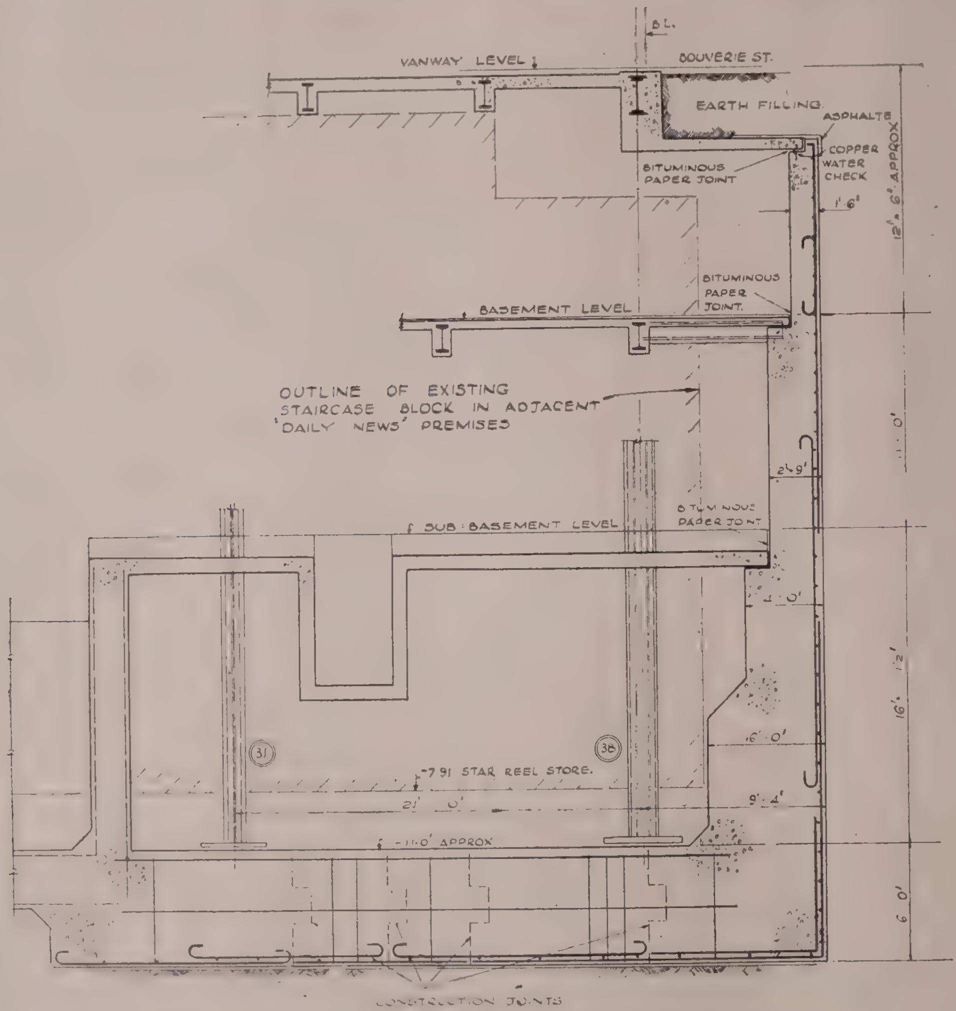


Fig. 6

Fig. 6a (*Photograph by courtesy of* DAILY NEWS LTD.)

sufficiently large enough to insert the grillage to support stanchions 26 and 27. Finally this grillage was manœuvred into position under the existing foundation and the stanchions attached thereto as shown in Fig. 7b. After levelling and grouting, the whole unit was concrete cased to within 3 in. of the top and then pinned up as described previously. The external portion of the steel needles were cut off by oxy-acetylene flame, the remainder of the needling and R.S.J. posts being left cast in solid with the foundation unit.

A further foundation problem of a different type presented itself at the eastern section of the south boundary adjacent to Magpie Alley. Along this eleva-

(*Photograph by courtesy of* DAILY NEWS LTD.)
Fig. 6b

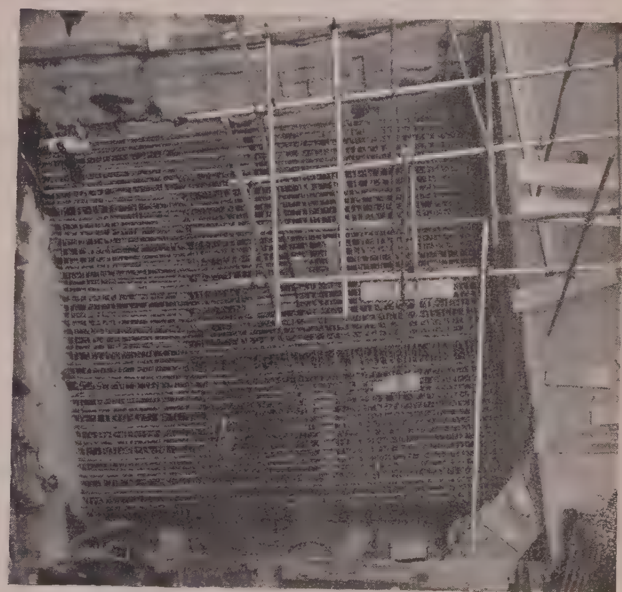


Fig. 7a

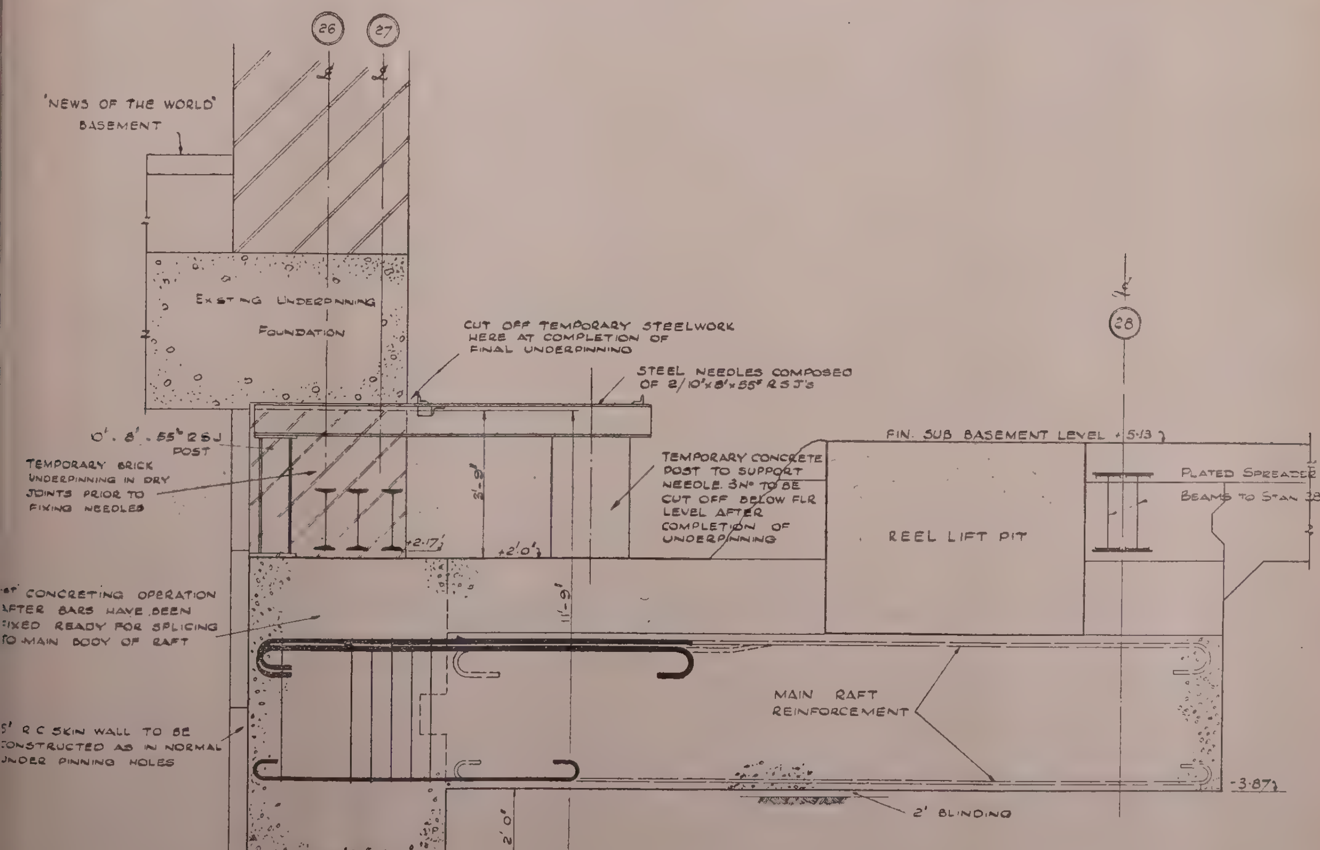


Fig. 7

tion, the outside of the structure was to be supported on the existing basement wall to the Associated Newspapers' premises, and at this point, Magpie Alley forms a roof to the basement of this building. During the years 1923-1925, when the aforementioned building was erected, a steel-framed retaining wall was constructed which is approximately 50 ft. high and formed underpinning to the earlier DAILY NEWS building. In addition to this the basement to the original DAILY NEWS building had been lowered subsequent to its original construction. This latter work was carried out prior to the construction of the Associated Newspapers' premises and the wall and formed, to support the roadway forming Magpie Alley. Since this latter thoroughfare is a public highway, it had to be continuously maintained. After the opening had been formed, the top of the existing concrete retaining wall was trimmed off and a concrete bearing pad was formed. The new engineering brick wall was then built for the length of the opening to the required height, with toothing left to bond in the subsequent 5 ft. widths. Each subsequent unit was constructed as described until the whole length was completed. Finally, the stanchions were founded on this wall and transferred their load to it by means of steel base plates,

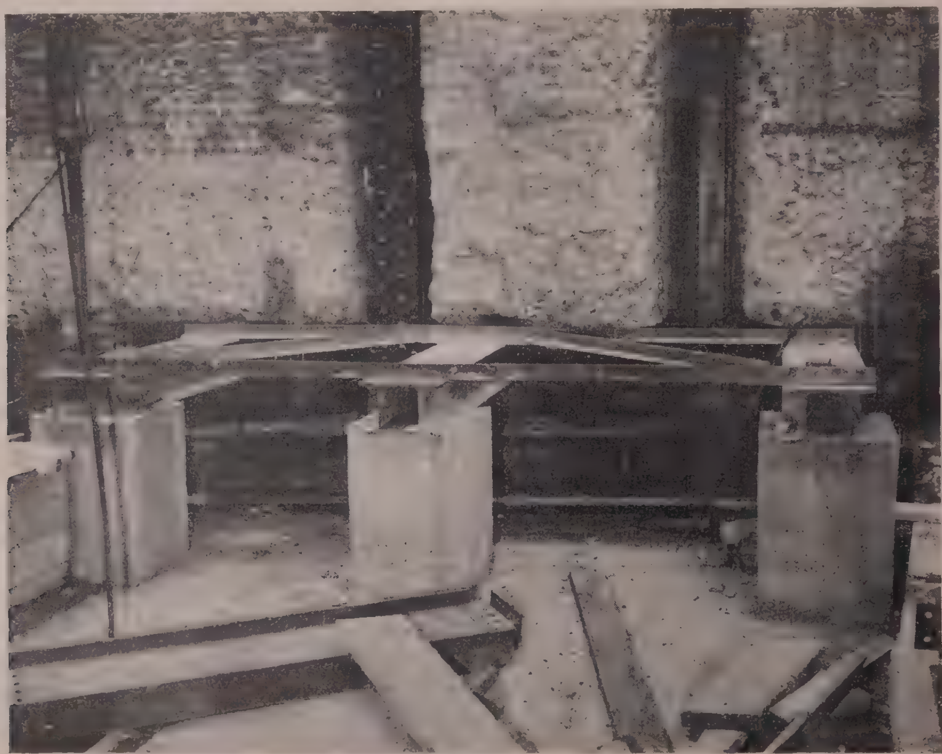


Fig. 7b

foundations described were further underpinned between the steel-framed concrete wall and the original foundations.

In the new building four stanchions, Nos. 7, 12, 19 and 25, with base loads of 50 tons, 166 tons, 227 tons and 259 tons respectively, were to be sited on this wall. After general demolition the existing brickwork proved to be entirely unsatisfactory for supporting these loads and a new scheme was devised to meet the conditions. South of this wall and foundation were lockers and lavatories used by the Associated Newspapers' staff and these had to be maintained in use together with many main services in the same area. The only convenient and suitable medium for transferring these stanchion loads to the steel-framed retaining wall was engineering brickwork—laid as first quality work—capable of safely [illegible]taining 25 tons per square foot. The wall adopted [illegible] ft. $7\frac{1}{2}$ in. thick and, due to construction difficulties, [illegible] built in 5 ft. lengths. To carry out the necessary [illegible]ns, the first step was to contruct a temporary [illegible] in the locker rooms on the south side, thereby [illegible]g free and continuous access to the existing [illegible].

[illegible]g with one section at a time, 5 ft.-wide openings were cut in the existing wall (footings) and temporary steel beams were provided at high level across the opening the maximum thickness of which was 9 in. Fig. 8 shows the nearly completed wall.

Framework

The whole of the superstructure was carried out in structural steel with solid reinforced concrete floors. Like kindred buildings in the newspaper industry, the main endeavour in the planning is "speed and efficiency" with the result that there is a large number of intermediate floor levels and seemingly uncommon structural features.

To form the main machine room at basement level at the rear of the building a clear space was required 70 ft. long × 37 ft. 6 in. wide × 26 ft. high. The main structural units forming the ceiling to this are two double and one single web plate girders, weighing a maximum of 21 tons each. A further girder of 37 ft. 6 in. span × 6 ft. 10 in. deep under the foundry room floor and over the same area was designed with four access holes through the web—this girder weighed 27 tons and is well illustrated in Fig. 9. The longest span girder in the framework is at the first floor level over the vanway. This member is 53 ft. 6 in. long × 4 ft. deep and is of plate and angle construction weighing just over 20 tons. In the completed structure this girder was cased in concrete and faced with Portland stone. In arranging

a programme for stone layers for this work, attention was drawn to the anticipated deflection of this girder when fully loaded. The total deflection was calculated to be 1.20 in. and it was considered that if the stone facing was to be fixed before the full application of superstructure dead load, then there would be a likelihood of the stone cracking as the beam deflected. To over-

this point upwards a series of cranked frames were introduced on these three elevations at each subsequent floor level up to roof.

To meet the clients' needs, many accesses were formed between the new and existing adjacent buildings, including a bridge at second floor level on the south face. In constructing the latter, an opening 12 ft. high ×

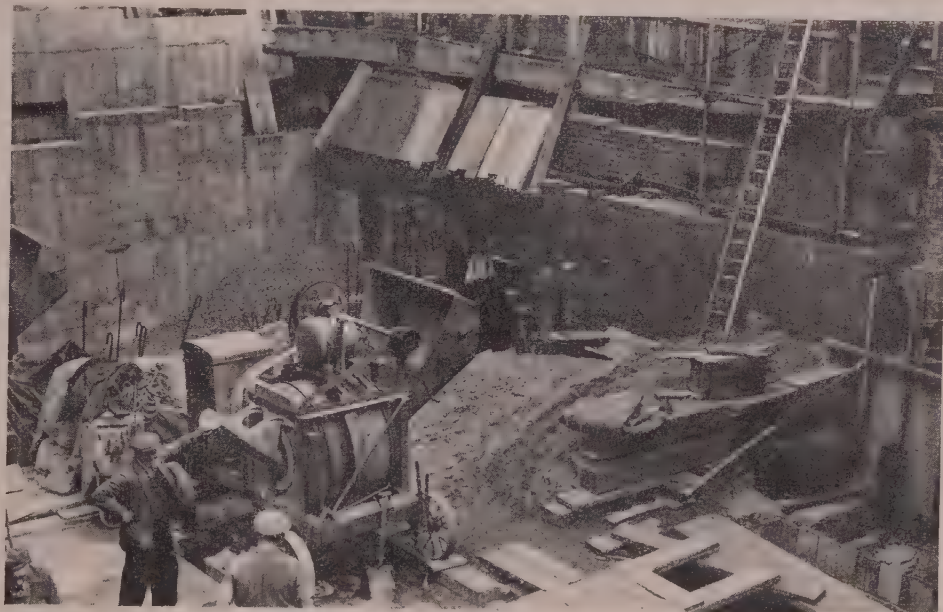
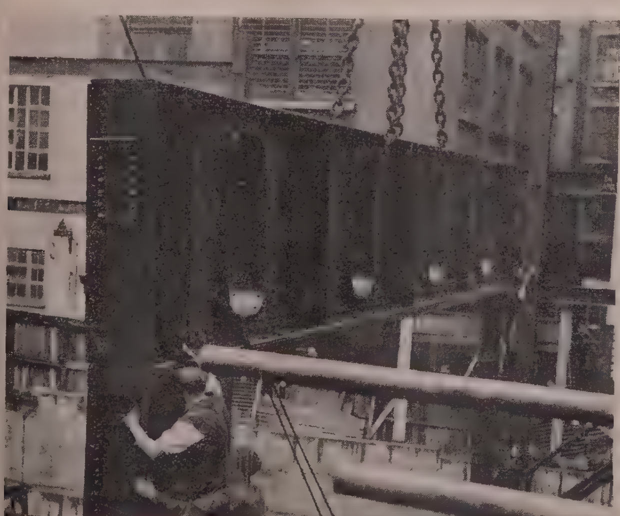


Fig. 8 (*Photograph by courtesy of* DAILY NEWS LTD.)

come this possibility a programme was arranged whereby the whole of the superstructure, floors and brickwork was completed before the facing was fixed. This ensured that the beam realised some 65 per cent. of its gross loading before final cladding operations began and the risk of damage to the stonework was considerably



(*Photograph by courtesy of* DAILY NEWS LTD.)

Fig. 9

lessened. As mentioned in the earlier paragraphs, the structure is raked back above cill height at third-floor level on the north, south and east faces to meet statutory requirements with regard to light and air to buildings. This arrangement involved nearly all the external stanchions being terminated at second floor level and from

11 ft. wide had to be formed in the external wall of the existing building. This was a heavy load bearing wall and a complicated form of strutting and needling was designed and constructed so that the necessary cutting away could be effected in the old building. On completion of this preparatory work a structural steel box frame was erected in the opening and the brickwork above and below pinned up. After removal of all strutting and needling, the erection of the bridge followed, which was carried out in conventional slab and beam construction.

Conclusion

The whole of the demolition, construction of the foundations, steel frame, reinforced concrete floors and dormer slopes was completed in two years and three months. The total amount of structural steel used in the superstructure amounted to 700 tons and 190 tons of mild steel rod reinforcement in the whole of the reinforced concrete work.

Throughout the whole contract there were many associated people whose contribution to the total scheme was invaluable and the authors would record their acknowledgements to Mr. T. W. Parsons, A.M.I.Mech.E., M.I.Struct.E., M.Soc.C.E.(France), Director and Chief Mechanical Engineer of the NEWS CHRONICLE and THE STAR, who was primarily responsible for the production planning and the original conception of the building ; Ellis, Clarke & Gallannaugh, F/F.R.I.B.A., who were responsible for the general design of the building and all services ; Trollope & Colls, Ltd., the General Contractor, who carried out the construction ; Moreland Hayne and Co., Ltd., Structural Steel Engineers, who fabricated and erected the structural steelwork, and Mr. Frederick S. Snow, O.B.E., M.I.C.E., M.I.Mech.E., P.P.I.Struct.E., who acted in an overall advisory capacity, and encouraged the authors to prepare this paper.

Fatigue of Welded Structures*

Discussion on the Paper by Dr. R. Weck, A.M.I.C.E., A.M.I.Mech.E.

The CHAIRMAN proposed a vote of thanks to the Author and invited discussion on the paper.

Discussion

Mr. G. M. BOYD (Member), in opening the discussion, said that he must congratulate the Institution on a first-class paper. They had been very fortunate in persuading Dr. Weck to give his address.

What had struck him forcibly in reading the paper was that Dr. Weck and himself had frequently been bracketed together as apostles of doom who went about unnecessarily spreading alarm and despondency among engineers, Dr. Weck on fatigue and himself on brittle fractures. For himself, he made this admission with a certain amount of pride, since the first necessity was to reveal the truth, and the second to face up to it and act as it demanded. He quoted the final paragraph of the paper, which was not merely " rounding off " but sound and considered comment, as follows : " The difficulties in the design of welded structures subject to fatigue loading are a consequence of the versatility of the process. They present a challenge to the ingenuity of the designer, the fabricator and the man in the laboratory to use this versatility in overcoming them. *They cannot be overcome by ignoring them.* Nor can progress be made and experience be gained if for lack of courage and enterprise we refrain from using a most advantageous method of construction because there are certain difficulties and complications when fatigue failure cannot be safely ignored."

Every word of that applied to all the other problems of structural engineering. In that respect, his apology for spreading the supposed unnecessary alarm and despondency was that nothing was to be lost by citing the truth, but much could be gained by it.

On the subject of alarm and despondency, one thought which came out of the whole paper was the fact, just emerging, that the ideas of stress on which structural engineers had all been brought up, and on which most of their calculations were based, was an inadequate concept to contend with some of the phenomena that were becoming increasingly important. Dr. Weck had shown admirably the way in which ordinary stress concepts were quite misleading in connection with fatigue, and the same thing occurred in connection with brittle fracture.

The time was coming, if it had not already arrived, when the whole basis of thinking on structural design must be overhauled, not only on the question of stress but on the statistical aspect. Whether the realisation was palatable or not, it had to be realised that no matter what one did, there could never be a structure which was 100 per cent. safe for an indefinite period. The problem was to reduce the risk to an acceptable level.

He wondered whether Dr. Weck would scotch or correct the feeling, which was very prevalent, that when a metal had been subjected to fatigue, the metallurgical structure was in some way damaged in the sense quite commonly expressed that the material had become " crystalised " or " fatigued."

Stress corrosion was another of the bugbears that was coming out. It was particularly insidious, because unlike ordinary corrosion, which could be seen on the surface and could be dealt with and protected in some way, stress corrosion did not show anything much on the surface until it became dangerous. It became either a precipitate fatigue fracture or a brittle fracture.

He thought there were printer's errors on page 120 in the section dealing with high tensile steel, the last part of which was extremely important. In contemplating high tensile steel one might envisage a higher factor of safety, but this might be lost owing to fatigue. He gave the warning that some high tensile steels might also be susceptible to brittle fracture, unless proper precautions were taken.

A thought which had occurred to him during Dr. Weck's introductory remarks in connection with aircraft structures and double booms was whether it would not be possible to put in a dummy member which would be likely to fail by fatigue a little earlier than some other important member—in other words, to form a kind of safety valve, so that when that part went one looked out for trouble and grounded the aircraft.

Dr. WECK in reply, said it was nonsense to speak of fatigue failure as if it were a consequence of the material having been " crystallised." Metals are crystallised as soon as they solidify from the liquid state. The cast crystal structure is modified during hot or cold working processes such as rolling, drawing extruding, but there is no evidence that any significant change in crystal structure is produced by many applications of loading. The view that metals crystallise in consequence of fatigue loading survives from the time, over a hundred years ago, when it was not realised that metals are crystalline anyway, even in their virgin state. The surface of a fatigue fracture generally exhibits two distinct zones. One, the fatigue failure proper is of smooth, velvety appearance and often exhibits arrest lines, that, is more or less concentric markings spreading outwards from the nucleus of fracture. When the fatigue failure has spread far enough to weaken the cross-section sufficiently that it can no longer support even one loading cycle the fracture will suddenly extend over the rest of the cross-section and this " final fracture" surface generally exhibits an entirely different appearance, it looks coarsely crystalline and is in fact what we now call a " brittle fracture." The areas occupied by the fatigue failure and the brittle " final fracture " depend on the type of material, the conditions of loading and the temperature at which the fracture develops. In some cases the brittle part of the fracture occupies a very large proportion of the area and it is this phenomenon which must be considered responsible for the erroneous view that the crystallinity of the fracture appearance was a consequence of fatigue.

Stress corrosion cracking to which Mr. Boyd refers must not be confused with corrosion fatigue to which reference is made in the paper. Stress corrosion is a phenomenon which under certain circumstances will occur in metals subject to purely static stress either externally applied or residual. The season cracking of brass cartridge cases, the caustic cracking of riveted boilers and the cracking of crude gas mains are exam-

**Read before the Institution of Structural Engineers at* 11, *Upper Belgrave Street, London, S.W.*1, *on April* 22*nd*, 1954. *The President, Lt.-Colonel R. F. Galbraith, M.C., B.Sc.(Eng.), M.I.Struct.E., A.M.I.C.E., in the Chair.*

ples of this phenomenon. Corrosion fatigue on the other hand is exactly the same phenomenon as so-called "dry fatigue," except that the process is greatly accelerated, which means that in the presence of a corrosive medium fatigue failure will start earlier than it would in the absence of corrosion for the same applied stress or, to put it another way: whereas a certain stress level may be endured indefinitely often in the absence of a corrosive medium, fatigue failure will occur if corrosive influences are present.

There are in fact printer's errors on page 120 which were the result of the printer misunderstanding the corrections made in the galley proofs. It should read:

	M.S.	H.T.S.
in tension from zero to maximum	16.6 tons/in.^{2}	22.2 tons/in.^{2}
in alternating compression tension	$\pm$10 tons/in.^{2}	$\pm$12 tons/in.^{2}

The idea of the safety valve was interesting. The riveted structure had a safety valve of a kind. If riveted joints were severely overstrained under fatigue loading rivets worked loose. Fatigue cracks might then not occur if the structure was such that a certain amount of stress redistribution could occur so that members over-stressed from the point of view of fatigue could shed some of their load to other members, particularly if the structure was highly redundant. He believed that this "load shedding" occurred also in welded structures after fatigue cracks had actually started provided the structure was highly redundant. In welded ships for instance fatigue cracks might start, develop to a certain length and then stop because the development of the crack has produced load shedding in the particular highly stressed region.

Mr. O. A. Kerensky (Member) said that he believed Dr. Weck was often disappointed by the welding details produced by designers. He was alarmed by the paper, because it left practically no details for the designers to produce that were satisfactory from a fatigue point of view: no fillet welds, no flange to flange welds, no welding across the line of stress, no egg crating and slotting of plates, no intermittent welds; and even corner gussets were of no help in reinforcing joints. The only things left were the good old bolts and rivets, as the author admitted on page 120. Unless they could separate the problem of fatigue design from the problem of ordinary design and establish quantitative criteria for welded details, the welding design would become almost impossible.

They should now try to consider the four categories of design. First, clad buildings, in which he thought one could forget fatigue. They were all familiar with the horrible notched webs, cleats, stools and fish plates used in riveted design. Surely nothing worse could be invented for welding.

Secondly, unclad buildings, practically every one of which, he thought, would not be subject to fatigue, except perhaps machinery housing, crane gantries and certain towers.

The third category was highway bridges. The author had said he did not think that a highway bridge was likely to suffer from fatigue, with which they would all agree; but Dr. Weck had qualified this by saying that, "it depends largely on detail." This put everyone back to where they were before. If there were introduced fillet welds across tension flanges of plate girders, which was practically unavoidable in ordinary commercial design, or if a plate was slotted for continuity effects, as was frequently done in riveted design, or if use was made of the joints (c), (d), (e) and (g) shown on page 123, and (a) and (b) on page 127, was fatigue still to be ignored in a highway bridge, or must it now be considered? He hoped that they could still ignore it.

Alternatively, would it be possible, even at the cost of a considerable amount of experimental work, to impose a definite limitation—for example, to have fillet welds across tension flanges of plate girders, and to say that the weld was safe for ever at say from 0 to 5 tons per sq. in. for mild steel and from 0 to 7 tons per sq. in. for high tensile steel? He knew that the scientist refused to be tied down to figures, but what was the designer to do? Unless some such simple cut-off was introduced and one was told that in a certain stress range there was no need to worry about the presence of fatigue-raising details, they were not escaping fatigue problems in highway bridges.

The next category was railway bridges, which Dr. Weck had suggested might be definitely subject to fatigue. At the Welding Conference in November, three prominent railway engineers argued that even in this case it was quite unnecessary to take fatigue into account, because maximum loading impact, lurching, and wind, were seldom present together, and that consequently the normal everyday load was considerably smaller than the design load.

He was inclined to agree with Dr. Weck that in certain circumstances some members of a railway bridge were liable to suffer from fatigue if such a thing as fatigue existed at all, and that therefore fatigue must be allowed for.

When dealing with riveted joints, however, they took it for granted that a riveted joint designed to say $\pm$6 tons per sq. in. cycles was practically good for ever. The number of failures of a well designed joint was very small. Could something like this be done about the welded joint? With a certain type of joint—say, a flange to flange at right-angles—could some kind of figure be put on it, even if it was as low as 0 to 4 tons per sq. in.? If so, the designer could try to estimate the number of repetitions of the heavy and of the light loadings.

Another question which had been partly discussed was whether fatigue effects were cumulative. That was to say, if a detail would stand, say, 600,000 pulsations of 0 to 9 tons per sq. in., would it stand additionally, say, 10 million pulsations of 0 to 3 tons per sq. in.? The figures were immaterial, but could Dr. Weck say whether, having estimated that the heavy load would occur only a certain number of times, they could safely forget it and add to it over the life of a 100 years the normal small loads?

He concluded from the paper that the truss was almost impossible to design for fatigue resistance. The Swiss joint, shown by Dr. Weck, which was produced in Zurich some years ago, was a very expensive one to make, tricky to assemble, and he did not himself think it had any future. He did not think the introduction of cast steel or forged steel junction pieces had much future either. The cast steel would have to be radiographically tested and the price would be over £300 per ton. He wondered what happened to tubular construction and joints of pipe to pipe as far as fatigue was concerned?

As Dr. Weck rightly said, riveted structures designed at 7 to 9 tons per sq. in. had not suffered unduly from fatigue. If a fatigue failure occurred, it was not usually catastrophic, because the crack was arrested by rivet holes somewhere. Welded structures without holes should be much better, but they could also be much

worse and the crack would be catastrophic. Hence the prominence of the fatigue problem in welded construction.

Having listened to probably one of the greatest experts on fatigue in England, he had regretfully come to the conclusion that the position was still most unsatisfactory. A great deal was known about fatigue, but many more facts would have to be established before the designer could tackle the problem rationally.

Dr. WECK replying, said Mr. Kerensky in his infallible manner had put his finger on all the weak spots of the subject. He was sorry to have created the impression that no details were left. Perhaps Mr. Kerensky was exaggerating somewhat. Even if one could not use fillet welding in the most highly stressed areas and even if one had to dispense with "egg box" construction which was not economical in any case, this did not mean that no details were left. It merely means that where fatigue is of importance the designer cannot slap bits and pieces together in a happy-go-lucky way as was so often done in welded construction.

However, even if there were no details left he could hardly be blamed for it. Their job was to find out with the means at their disposal, that is, available laboratory equipment, what they thought were the facts. Certain results were obtained and it rested with the designer to decide to what extent these results were relevant. The designer was quite at liberty to disregard all laboratory work and in that case one could only wish him luck.

Designers were reconciled to the idea that in stability problems the exact load factor was often impossible to assess because the particular case could not be solved mathematically. Stability, as every designer knows, is an extremely complex problem. Fatigue was equally complex, and he (Dr. Weck) could not see why designers should be reconciled to complexity and approximations in stability problems, whereas they insisted on simplicity and accuracy when it came to fatigue, which was in many ways an even more complex problem. It is impossible to make general statements without qualification. Highway bridges were not likely to suffer from fatigue even in ordinary commercial design but it must be realised that even in highway bridges there will be a limit beyond which it would be unwise to push simplification in design which was made so very easy by welding. The answer to Mr. Kerensky's question is surely that fatigue in highway bridges can be ignored in the sense that no reductions in permissible stresses are necessary provided that reasonable care was exercised to avoid the most serious types of stress raisers such as butt welds with only partial penetration, joints in members with flange straps only and so forth. It was impossible to enumerate all the possible mistakes a designer could make if he ignored fatigue altogether. The important thing was to realise that highway bridges too are subject to many repetitions of loading though of small amplitude and that one cannot hope to get away with the same sort of detail that was satisfactory for a purely statically loaded structure.

The scientist does not refuse to be tied down to figures [illegible]ided he is given the chance to ascertain the figures [illegible]y experiments. No fatigue tests on plate girders with [illegible] welded to the tension flange have been carried [illegible] this country and such scanty information as is [illegible] from abroad does not permit to draw the [illegible]t-off Mr. Kerensky so much desires.

[illegible]ed with Mr. Kerensky that it would be desir[illegible] a figure on certain types of joint such as, for [illegible], a flange to flange at right-angles. To do this required experiments and perhaps Mr. Kerensky could [illegible] the industries interested in sponsoring some research to get more information on this and some other structural problems. One of the reasons why many of the questions Mr. Kerensky asked could not be answered was that the structural industry had so far shown little interest in fatigue investigations on the assumption that this did not concern them.

The question whether fatigue effects were cumulative must be answered in the affirmative but this is about as far as one could go on the basis of knowledge at present available. The question whether a detail could stand 10 million pulsations from 0 to 3 tons/in.^{2} in addition to, say, 600,000 pulsations between 0 and 9 tons/in.^{2} could simply not be answered on the basis of present-day knowledge. In order to answer this question one would have to know the S-N curve for the particular detail. If one then applied the simple cumulative damage rule he, Dr. Weck, felt one came pretty near to the truth. The simple damage rule postulates that the damage of a single cycle at a particular stress level is inversely proportional to the total endurance at that stress level so that if n_1 cycles at stress S_1, n_2 cycles at S_2, n_3 cycles at S_3, etc., are applied where the total lives for S_1, S_2 and S_3 are N_1, N_2 and N_3

$\frac{n_1}{N_1} + \frac{n_2}{N_2} + \frac{n_3}{N_3} = 1$. However, it should be said that there is some strong evidence to show that this simple rule is true only very approximately. There again, he could only say that if the structural engineer considered that this was an important problem, an Institution such as the one he was addressing was the body which should do something to initiate work to produce the information.

There is very little known about the fatigue strength of structural joints in pipes. There is no reason to suppose that they would be any better than joints in other structural members. If anything they might be expected to be somewhat worse because without backing rings it was somewhat more difficult to make a full penetration butt weld in a pipe than in, say, a channel section, and even for fillet welds it was somewhat more difficult to achieve the accurate fit up of parts essential for first-class fillet welding, that was relatively readily achieved in joining flat surfaces. It was only right, however, to recall that remarkable tubular structures subject to fatigue loading had been built which as far as he knew had given no trouble. The 280-ft. tubular boom of the large walking dragline built by Ransome and Rapiers for Stewart & Lloyds at Corby is a notable example. As far as he (the author) knew, this structure so far has been free from any fatigue trouble.

Professor A. G. PUGSLEY, O.B.E. (Vice-President) drew attention to the many contributions made by Dr. Weck in the field of fatigue beyond those referred to in the paper.

He had been tempted to speak because of the references in the paper to fatigue in aeronautics, the fatigue problems of which he happened to know better than those of ordinary structures. There were a few points which had been touched on in part and about which he would like to ask questions. The first related to the author's blackboard diagram.

Aeronautical people nowadays happened to be the people most worried about structural failure, and at the commencement of a new aeroplane design they tried to estimate its life in terms of flying hours. Unfortunately, the lives of modern aeroplanes were only too short commercially; the average in recent years was probably about 10,000 flying hours.

When presented with this problem, the designer of a new aeroplane naturally did fatigue tests on the proposed joints and other critical parts of his structure. When the tests showed a very wide scatter, he must take a conservative view and estimate, perhaps, that the aeroplane would be assuredly safe for only the first 5,000 flying hours. If he stopped there, the airline or other commercial user would be very dissatisfied ; the aeroplane had apparently to be scrapped after 5,000 flying hours, which mounted in a year or two. There was, therefore, great pressure for further investigation.

The only way at present available for proceeding further, as he saw it, was the way that Dr. Weck had suggested of taking out from service certain of the critical parts, which were tested earlier in their initial condition, to test them again and compare their lives under the same conditions. This was being done at present both in Britain and in America, and the net result was commonly to allow the aeroplane to live for another few thousand flying hours ; thereafter some replacements might be necessary. Sheer commercial pressure was thus causing research testing.

Did that type of test show signs of being hopeful or possible with welded joints ? In this matter, he was with Mr. Kerensky. Aeronautical people commonly avoided welded joints because of their fear of fatigue troubles. There were, of course, other reasons as well.

His other question had also been touched on in part. All real structures, as distinct from parts of machines, did not undergo regular cyclic loading. They underwent a most irregular history of occasional loads and load fluctuations. Even under railway bridge conditions there were irregular loads and different loadings, different times between loads, and so forth. The aeroplane was not unusual in this and it had a history of loading which was difficult to predict. It had to be followed statistically by measuring the loads on aeroplanes going backwards and forwards day by day.

Most of the more critical aeroplanes were equipped for this purpose, so that there was a load history known for the aeroplane. This information had to be related to a system of testing. Fatigue testing had grown up around cyclic loading. To put it through any other cycle was extraordinarily expensive and difficult. As a result, there was great need for knowing the equivalent mechanical cycle to replace or represent the real history of loading. This had at present only one answer, with which most scientists—and probably Dr. Weck—would strongly disagree for any individual case, but it was the only method available to aeronautical engineers.

It was what was called the " cumulative damage rule," originally proposed in America by Miner.* It enabled one from an irregular load history to pick out from that history, in popular terms, the most damaging sinusoidal cycle that was present. The most damaging cycle having been chosen, the joint was tested in the ordinary way, using that cycle. He asked whether any work in the welded joint field had been done on the applicability, or the reliability or unreliability, of anything like the " cumulative damage rule."

In relation to railway bridges, he thought there was a chance of getting at an equivalent load cyclic load for its cross girders. The aeronautical man had had to clutch at the cumulative rule, not because he believed it in detail, but because it was the only way he could get a shot at the main source of trouble. He found it was not the occasional loads or high gusts that would do most " damage " ; it was the regular small oscillations of the wings, not in terms of feet at the wing tips, but in terms of inches. This was of great convenience experimentally, and so far as the evidence went it looked as if it was a fair shot. If the cross girders of railway bridges were looked at in the same way, presumably it was not so much the heavy loads on the locomotive axles as the continuous repetition of the smaller coach axle loads to which attention must be given. He wondered whether Dr. Weck would say anything about this.

Dr. Weck, replying, said that he felt Professor Pugsley was much better qualified to answer the questions he had asked than he, the author, was, particularly in the aeronautical field.

Taking a broad view of the fatigue problem in aeroplane and other structures, there were perhaps two approaches. The indirect approach which is customarily pursued and which consisted in first of all getting accurate information on the loading spectrum, followed by intricate stress analysis, followed by comparison of these stresses with the fatigue properties of the material determined with small specimens either smooth or notched in different ways. This he always felt was not a way holding out much hope of success. The direct way, that is the fatigue testing of critical joints and components, which in effect determine the fatigue resistance of the whole structure, seemed a more promising way. It certainly had been with regard to welded structures and he was glad to learn from Professor Pugsley that this was the method used by the aircraft designer. To his dismay he had recently seen an important paper by the Director, the Assistant Director of Research and the Assistant Chief, Structures Research Division of the National Advisory Committee for Aeronautics† in the United States, in which a test programme is outlined whereby it is hoped to predict the resistance of structures to repeated loading from the analysis of flight records (acceleration and gust loading) on the one hand and the S-N curves of smooth, perforated and notched aluminium alloy specimens on the other hand. This idea he had to confess did not impress him as being the right way of tackling the problem of fatigue in aircraft.

The type of test outlined by Professor Pugsley held out far more promise of success. For welded structures it may be rather more difficult to remove parts after a certain time in service for re-testing than it is for aeroplanes but the suggestion put forward by Professor Pugsley was worthy of very serious consideration.

Far from disagreeing with Professor Pugsley he, the author, was convinced that the cumulative damage rule proposed by Miner was at present the only practicable means for assessing the effect of loading cycles of differing amplitude. No work however had been done to show to what extent this rule could be reliably applied to welded joints. He quite agreed with Professor Pugsley that for cross girders in a railway bridge the many applications of smaller loads from coach axles, were more important than the smaller number of heavy loads from the locomotive axles.

From what he had seen of aeroplanes—which admittedly was not very much—he had the impression that the details of aeroplane structures had often been designed as if fatigue could be entirely ignored. If as much attention were paid to the avoidance of stress concentrations in the design of details as was paid to intricate methods of mathematical stress analysis of aircraft structures, the aircraft industry might not be experiencing as much trouble now as they seemed to be.

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*JNL. OF APPLIED MECHANICS, September, 1945.

†The Fatigue Problem in Aeroplane Structures, by H. L. Dryden, R. V. Rhode and P. Kuhn.

Fatigue and Fracture of Metals. A symposium held at M.I.T. 1950, Chapman & Hall, London, 1952.

Mr. S. M. Reisser (Member) said that he wished to raise a simple question arising from page 124 of the paper, in which Dr. Weck said : " Hence, the practice of not welding stiffeners to the tension flange of a plate girder."

In this country British Standards do not create but are supposed to represent what is good standard practice and although Clause 47 (b) expressly forbids the welding of the stiffeners across the tension flanges in cases of dynamic loading—welding parallel to the length of the flange is allowed. This requirement follows Continental practice and he did not think it was a good idea, since fatigue cracks always started from a *point* of maximum stress. He could not see what difference it made whether there was one such point or a line across the flange and perhaps Dr. Weck would say whether he was satisfied with this clause or whether it ought to be revised.

The diagram on page 119 showed that there were fatigue failures in compression. He was aware, of course, that tests had been carried out and that fatigue failures in compression did occur, but he was under the impression that the number of such tests was small. He wondered whether the basic compression stress B″—B″ in Fig. 3 was equal to the basic tensile stress A″—A″, and how the diagram was obtained—i.e., whether from test results or from some form of deduction.

On page 126 Dr. Weck referred to extension pieces, and said that they should never be tacked to the flanges. Incidentally, this was the easiest and the usual way to put them on. Did Dr. Weck object very much if the tacking was done inside, so that it was later incorporated in the rest of the weld ?

He differed strongly with the author's statement on page 124 that it was good practice to design joints so that they were slightly stronger than the members themselves. This was the practice, he believed, in some members—in most, perhaps—in bridge work, but it certainly was not the practice in structural work. It would be a waste of money to design joints to transmit more than the load in the member, since the size of the member may have been dictated by reasons which had nothing whatever to do with the load it was carrying. The standard practice was to design a joint to carry the load which it must transmit, and he saw no reason why this should be altered.

Dr. Weck replied that B.S. 449 referred to building structures, which, as Mr. Kerensky said, were not subject to fatigue. So when designing under B.S. 449, they could weld their stiffeners on, leave them off, or do whatever they liked. In designing a plate girder, which was definitely subject to fatigue loading—and in this case he doubted whether it should be designed to B.S. 449, he thought it was definitely better not to weld the stiffeners on. The German practice of welding them on with a longitudinal rather than a transverse weld was no better—and, in his opinion, might be worse—than a transverse weld. There were very serious stress

concentrations at the end of the longitudinal welds, and they were no less deleterious than a transverse fillet weld. But he did not see why anyone should want to weld stiffeners on particularly in the tension flange unless it was absolutely necessary.

The clause in B.S.S. 449 to which Mr. Reisser referred (clause 47b) should be revised.

He did not say that any plate girder subject to fatigue where the stiffeners were welded on would invariably fail, but from what was known about the problem the probability of failure in a plate girder subject to fatigue if the stiffeners were welded on was higher than if they were not welded on. It was really a question of probabilities.

He was opposed to the tack welding of extension pieces, for reasons which were explained in the paper. It left end craters from which, in his own experience, fatigue failures might start. There were no objections to welding on the extension pieces inside in the weld groove, which would be welded over afterwards.

That might be, as Mr. Reisser had said, the simplest way, but the simplest way in welded construction from the point of view of fatigue was not always the best or most satisfactory way. It was equally simple in constructing a plate girder to make a very simple jig to hold the extension pieces on.

It was to be deprecated from the point of view of fatigue to have broken tack welds all over the place. Tack welds, even if ground off, like any other kind of weld, reduced the fatigue resistance and fatigue failure could start from tack welds as easily as from other welds.

Mr. Reisser had completely misunderstood his remarks about joints for static loading being stronger than the material joined. He had not claimed that these joints were designed intentionally to be stronger, but by the normal design rules used for the design of butt welded and fillet welded joints under static loading they invariably were stronger if the main material was designed for the maximum permissible stress.

Mr. Reisser would probably agree that in making a butt weld between two plates, where the thickness of the butt weld was equal to the thickness of the plate, where the gap was sealed properly and where there might be a little reinforcement on the top, under tensile loading failure would take place in the plate and not the weld. Similarly, in testing say a fillet welded lap joint where, say, two plates were attached by fillet welds to a third plate, and where the fillet welds were designed on the basis of say B.S. 449, it would be found that fracture would take place normally in the parent plate and not in the weld if the joint was tested to destruction under static tension.

That meant that a joint designed on the basis of B.S. 449 was in fact stronger than the material joined, because the material broke first under test.

Mr. Reisser pointed out that it depended on the size of the fillets.

Dr. Weck said that the size of the fillets must always be 40 per cent. more than the parent metal. The parent metal was designed for a stress of 9 tons per sq. in., and fillet welds for 6 tons per sq. in. If a quarter fillet was used, it must be made much longer. A certain throat area had to be provided to transmit a certain load. The throat area which had to be provided in a fillet weld joint, on the basis of the permissible stresses at present in use, was 40 per cent. greater than that which was joined. If the joint was tested like this, it would fail in the plate, which meant that *de facto* the joint was stronger than the material joined.

Mr. Reisser asked whether, if there was a member one inch thick carrying only 3 tons, the fillet would be made strong enough to develop the strength of the one-inch plate or simply to take the 3 tons.

Dr. Weck replied that if there was a member which for some extraneous reason was not subject to a load-producing maximum permissible stess the joint would

be designed for the actual load to be transmitted and in this case the joint would be weaker than the plate, but only because for some extraneous reason the plate was made rather bigger than it need be. Normally, this was not the case because the designer would endeavour to use the material everywhere to the maximum possible permissible stress. A continuous bottom boom would be stressed at the ends to a smaller value than at the centre but if it was continuous there would be no joints.

Mr. REISSER asked, "What about the ends"?

Dr. WECK said that if there were joints at the ends in a truss between diagonal members and the boom, the joints would have to be designed to take the full load from the diagonal. If the diagonal was fully used at 9 tons per sq. in., there would be 40 per cent. more throat area than in the parent metal.

Normally, one would have joints in the material which was fully used. A joint was made to change a section. A section was changed because at the particular point the stress had decreased. In the case of, for instance, a tension flange in a plate girder, with a continuous flange there was no joint in it; but if there were joints in the flange, they were made because one wanted to stress all parts of the flange to 9 tons per sq. in. He did not think there was any difference in his and Mr. Reisser's view. He had not advocated, as Mr. Reisser suggests, that it was good practice to design joints so that they were stronger than the members themselves. It just so happens that the joints are stronger than the member if both the member and the joint are designed to their full permissible design stress.

Fatigue failures in compression occur although the compression fatigue strength is generally higher than the tensile fatigue strength; the value B″—B″ in Fig. 3 was generally greater than A″—A″. The diagram as drawn was merely schematic; it would have to be obtained from actual fatigue test results. Whereas the fatigue strength in compression was higher than in tension if the compression flange of a beam was reinforced by a strap extending over, say, the middle third and attached by fillet welds fatigue failure would then probably occur at the end of the reinforcing strap in the compression flange and not in the tension flange.

Mr. FROST (Graduate) said he wished to refer to Dr. Weck's comment that the probability of failure due to fatigue might be greatly influenced by the design of the joint. As the joint was the point at which welding occurred, was it not possible that welding in itself had brought about a great increase in fatigue failures?

For example, in a butt welded flange plate the heat affected zone could be considerable in length. Over this length the size of the crystal in the metal could become very varied. As he understood it, the fatigue strength of a metal and also the transition temperature at which a brittle fracture might occur could therefore vary also. Might not these two things add up to increase the probability of fatigue failure?

Dr. WECK replied that the probability of failure in the joints was higher, not because they were welded. The probability would be also higher if the joints were riveted, glued or bolted, or held together by faith. The reason was that where there was a joint there was a discontinuity. Whether the joint was made by riveting or welding was, from a general point of view, immaterial. In fact, by the welding of mild steel one could make a joint of greater fatigue strength than by riveting. The welding itself as such, therefore, was not the "nigger in the woodpile." The difficulties arose from the changes in shape which normally took place at a joint.

If a butt weld was made, for instance, in a parallel flange, and the reinforcement of the butt weld was machined off, it would produce a joint vastly superior to a riveted joint in a flange plate, because the discontinuities which produced stress concentrations in the rivet joint were much more severe than those in the butt weld.

The question of a heat affected zone had nothing to do with it. For one thing, the heat affected zone in mild steel did not exhibit greatly inferior properties in any way to the weld metal itself or to the parent plate, proof of which lay in the fact that if there was a fatigue failure in a butt weld (if one was made experimentally in a laboratory) the fatigue failure might go through the centre of the weld. This happened when there was a root defect. The heat affected zone might be important in explaining the location of the fatigue failure at the edge of the weld and in explaining the reduction in fatigue strength of machined butt welds in comparison with unwelded plate.

If a fatigue failure was produced starting at the edge of a weld, it would not run along the heat affected zone but would run from the edge—the undercut—at right-angles to the stress. In mild steel, at any rate, the effect of the heat affected zone could be ignored.

In high tensile and alloy steels the problem was somewhat different, because without proper precautions there might be cracking in the heat affected zone to start with before any load was put on; but provided that high tensile steel was welded in such a way that there was a heat affected zone free from cracks, the fatigue crack would not run along the heat affected zone.

Mr. F. MICKLETHWAITE said it was rather distressing that the general picture of the discussion had been much to the disadvantage of welding from the fatigue point of view, and he would like to have Dr. Weck's further views.

His own impression on reading the paper was that far more was known about fatigue of welded structures than about fatigue of riveted structures. Was that perhaps not the reason why they were worried more about welded structures? Another aspect to be remembered was that with all dynamically loaded or repeatedly loaded structures, it was normal to have maintenance. Until they had sufficient records from maintenance engineers of failures of structures, both riveted and welded, they were perhaps forming a judgement too soon. They knew that there were quite a lot of failures in riveted structures which did not lead to catastrophic failure. Surely this was most likely to happen also with the welded structure.

Referring to Professor Pugsley's suggestion that aircraft engineers commonly avoided welded joints, he said the normal method of carrying the engine of an aircraft was on a welded tubular structure, which was the very point raised by Mr. Kerensky. He did not, however, know exactly the principles on which they were designed or what stresses were used. They were part of the structure which was subjected to repeated loadings of a severe order.

There was one aspect which had not been mentioned and on which he would like Dr. Weck's views. How were they to tackle the question of safety factors with regard to fatigue? He did not think it should be treated in quite the same way as for normal static loading and, taking a normal factor of, say, 2. It should be related to the life of the structure. He had no definite ideas on it and would be keen to hear what Dr. Weck had to say.

Coming down to fundamentals of fatigue, he did not think the paper said much about redundant or locked-in

stresses. It was known that in welded detail there were very high locked-up stresses, and a certain amount was heard about shake-down, and so on, but not a great deal. There also were rolling stresses, which could be of quite high magnitudes. Again, in welding-up plate girders, there were considerable stresses due to weld shrinkage, and so on. Furthermore, on welded trusses there might also be considerable secondary stresses arising from a similar question. Did allowances have to be made for this in design?

He raised the point because the draft specification in B.S. 153 contained a clause that with regard to fatigue stresses due to wind and temperature, secondary stresses should be ignored in considering fatigue. Clause 19 (a, ii), to which it specifically related, referred to secondary stresses which occurred repeatedly with heavy loading. He did not quite see why they should be ignored. The structure could be prestressed initially by deliberately building in stresses, thereby ignoring secondary stresses which came on in service.

Dr. Weck, replying, said Mr. Micklethwaite was quite right that fatigue failure was rarely catastrophic, as he had in fact tried to indicate in the paper. He felt sure that welded structures designed and executed with reasonable care and proper regard to the worst defects from the point of view of fatigue would give as satisfactory if not more satisfactory service than riveted structures.

It is perfectly true that in many aircraft welded tubular high tensile steel engine mountings are used but he doubted whether they were subject to fatigue loading in the same way as say the wing structure. Welded tubular engine mountings had been satisfactory although some fatigue failures had occurred. He did not believe that Mr. Micklethwaite's assumption was correct that these engine mountings were subject to repeated loadings of a severe order.

The concept of a safety factor is even more meaningless when fatigue loading is considered than for ordinary static loading. In fact, the concept of safety under fatigue loading must be arrived at on an entirely different basis. He, the author, believed that for fatigue loading a safety concept must be derived on a statistical or probability basis. All other safety concepts were completely devoid of real meaning for any structures or parts subject to fatigue loading.

The problem of the effect of locked-in stresses was rather complex and the author had intentionally omitted it from the paper. Briefly it could be said that there is no evidence to show that ordinary residual stresses due to welding or rolling had any influence on fatigue. There were, however, a number of contradictory experimental facts which it was difficult to explain. On the one hand the increased fatigue strength of shot peened leaf springs is ascribed by many people to the beneficial effect of residual stresses introduced into the surface by this process. On the other hand nobody has yet been able to show that there is a difference in fatigue resistance between identical welded structures or components of which one series was tested in the as-welded condition and a second identical series was tested after relief of residual stresses by heat treatment.

Reaction stresses perhaps were in a different category and might exercise some influence unless they were largely relieved under application of external load by plastic deformation. However, even if there was an influence it was unlikely to be very significant. Secondary stresses, that is stresses arising from eccentricity of connection, could influence fatigue resistance very appreciably. In fact the failures of American railway bridges, to which reference had been made in the paper, could be attributed to secondary stresses in as far as the fatigue failures occurred in floor beam hangers. In assessing the fatigue resistance of a structure the effect of secondary stresses if they cannot be avoided must be taken account of. The reason why secondary stresses due to wind and temperature are permitted to be ignored in the draft of B.S. 153 is simply that both wind and temperature stresses do not vary frequently enough to produce fatigue. To build in secondary stresses deliberately so that secondary stresses could be ignored in service would in general have no beneficial effect on fatigue resistance since what mattered was the total stress change at the extreme fibre and not the maximum stress value.

Written Discussion

Mr. Cedric Marsh (Graduate) writes: This revealing and comprehensive paper is no doubt intended to make us aware of the problem and dangers of fatigue as a potential cause of structural failure but perhaps the author is making us too aware of it. Where fatigue enters as a design consideration there is no doubt that it must be treated with due respect but it is a very small proportion of structures in which it does actually arise and, even in those, too much is often made of it.

To cite an interesting example, the main structure of an 18-ton platform trailer after some 30,000 miles of service, always carrying its full load, or a little more, was found to have fractured through certain rivet holes with some rivets failing to shear. The immediate suspect was fatigue but analysis of the design showed that these particular rivets carried virtually their ultimate load under normal working conditions. Fatigue would be expected to play a major role in the design of road vehicles as they are subjected continuously to loading cycles superimposed on a high mean load and yet rivets which should have failed under the load applied statically withstood 30,000 miles of fatiguing service.

If fatigue is to be the subject of specified requirement then the factor of safety applied on fatigue stress must be appreciably lower than those used for direct tension, as, by considering it, we are approaching more closely to the ideal state when factors of safety are no longer required.

The author's discussion of stress concentration overlooks the non-uniform stress distribution in a longitudinal weld. It is well-known that stress at each end of such a weld can be considerably higher than the mean stress. This high stress, normally ignored in design as it is levelled out by yielding before failure occurs under static load, could also be responsible for fatigue fractures. This would result in a crack along the weld whereas all references by the author are to fracture across the parent plate. It would be interesting to know whether in actual fact there have been such failures or whether in his experiences, all failures occur in the plate at the beginning of the weld.

As a corollary of this, if the high stress in each end of the weld does play some part in failure, then by increasing the length of the weld, this stress is reduced and the fatigue strength thereby improved.

In order for the stress to get from the plate into a longitudinal weld, as in Fig. 7d, there must be a natural concentration of stress in the plate towards the weld run, thus, even where no notch effect occurs, the direct tensile stress in the plate at the beginning of the weld will be higher than that in the plate remote from the weld and this will be more pronounced for short welds than long ones.

These points lead one to conclude that perhaps some benefit can be gained by increasing the length of the longitudinal welds although the notch effect at the toe of the weld may be the primary cause of failure.

In reply to written discussion, Dr. WECK writes :

It all depends on what one calls a " structure " when considering whether fatigue enters into the design of a " small " or a " large " proportion of them. If one takes in aircraft structures, excavators, cranes, vehicle frames, etc., one might say that as far as tonnage of steel or aluminium used is concerned, fatigue effects by far the largest proportion of all structures built including ships.

Mr. Marsh is certainly right in saying that if a factor of safety is used in design to safeguard against fatigue failure it can be very low. In fact if the stress producing failure after the requisite number of cycles were very accurately known there is no reason why the factor of safety should be greater than 1.

The example of the riveted trailer is interesting but in the author's opinion, who has had appreciable experience with all types of road and rail vehicles—unique. Fatigue failure of vehicle frames, particularly if the vehicles are mass-produced, is one of the most difficult problems to deal with.

The author is not quite sure what Mr. Marsh means by a longitudinal weld. If he means welds parallel to the direction of stress it is recognised that fatigue failure may originate at the ends of such welds ; it is rare, however, that such a weld in a structure has in fact an " end " which does not coincide with the ends of the parts joined. The important exceptions are intermittent fillet welds and reference to this has been made in the paper.

The author has never seen cracks along a weld caused by fatigue unless there was a stress—possibly not the main stress—acting at right-angles to the weld to account for this. A flange plate, generally assumed to be subject to uniaxial stresses is in fact subject to biaxial stress so that if a fatigue failure occurred along the weld between say the flange and the web it would be attributable to the transverse flange bending stress.

Once a weld exceeds a certain quite modest length no further reductions in the stress concentrations at the end of the weld are possible by increasing its length.

At the conclusion of the meeting, The PRESIDENT said that any further contributions by members would be dealt with by Dr. Weck in his written reply.

A vote of thanks to Dr. Weck for the way in which he had replied to questions was carried with acclamation.

Chairman's Address to the Lancashire and Cheshire Branch for the Session 1953-1954*

By Professor J. A. L. Matheson, M.B.E., M.Sc., Ph.D., M.I.C.E., M.I.Struct.E.

After thanking the Branch for doing him the honour of electing him their Chairman, and reviewing the improvement in the strength of the Branch since the war, Professor Matheson continued as follows :—

" Now I must turn to the main part of my task, which is to deliver a Chairman's Address. It is one of the happiest traditions of this Branch that these affairs should be brief, and this is one of those traditions which it is a pleasure to conform to.

" It is customary for Chairmen, of this as of kindred bodies, to talk about their own experiences in their inaugural lectures. As my own work lies in the academic field it is appropriate that I should say something about the *theory* of our subject. We often hear lectures about the practice of structural engineering and the interesting developments that are going on ; but it is very instructive to look behind the scenes occasionally, and notice what the ' back-room boys ' are doing.

" The history of the Theory of Structures records rather curious alternations of rapid progress and comparative inactivity. In the latter part of the 19th century, for example, Müller-Breslau and Mohr, the great rivals, stimulated by such practical problems as the difficult task of bridging the Rhine, carried what is now regarded as the classical theory to a high degree of perfection. They produced such an astonishing variety of theorems and techniques that their immediate successors must well have wondered whether there was anything left to discover.

" At that stage the theorists had provided the practising designer with powerful tools for dealing with the problems of the day. Graphical techniques, especially effective in the sphere of braced structures, gave reliable results even for statically indeterminate frames with several redundants. Influence lines had reduced the problem of the rolling load to a convenient and easily manipulated routine.

" But after the turn of the century the practical man began to forge ahead of the theorist. Tall buildings appeared on the skyline. Beginning in the over-crowded island of Manhattan, where the good earth is fortunately good rock, they have spread throughout the world, so that the city building of many storeys is symbolic of one aspect of civilised life as we know it.

" Then the economical French engineers began to experiment with that unpromising-looking substance—concrete—and to discover that it could be strengthened to an astonishing extent if steel were embedded in it in appropriate places.

" In neither of these cases was bracing very appropriate ; in building frames because of the objections of the inhabitants to diagonal members running across their offices, and in concrete structures because it is inherently simpler to make members intersect at right-angles than otherwise.

" When the new technique of welding invaded the province of the structural engineer the whole problem of the rigid frame was upon us, and for a time the analyst was pretty helpless. In 1915 Wilson and Maney, building on the foundations laid by Menabrea many years before, produced their slope-deflection method. But even this considerable advance left the designer with a formidable number of simultaneous equations to solve, and he rightly concluded that in

Given at Manchester on the 29th October, 1953.

most cases life was too short to embark on arithmetic on the grand scale.

"In 1927 an interesting invention was described by Professor Beggs, who had devised a method of analysing structures by making measurements on simple models. Here was something, one would have thought, which would have made an immediate appeal to the head of a drawing office—a simple, relatively cheap and convenient apparatus which produced answers of quite acceptable accuracy. Why did it not catch on?

"Well, there are probably several reasons. In the first place Beggs' own exposition leaves a good deal to be desired. The theory is not complicated; but those designers who had actually heard of Maxwell's Law of Reciprocal Deflections probably didn't believe it, and were most reluctant to base their designs on what was manifestly a piece of black magic.

"Teachers were not slow to see that here was a device of considerable pedagogic value, but the idea made almost no impact on the commercial design office in this country at least, until Pippard showed that Beggs' insistence on minute deflections was unnecessary and gave a more convincing account of the theory of models.

"But in any case new ideas take a measurable time to penetrate to the outer reaches of industrial practice, and before this could happen the redoubtable Hardy Cross struck a vital blow. His moment-distribution method is, of course, a brilliant idea and it lends weight to an interesting assertion; this is that new ideas, in advance of their time, fall by the wayside because the world is not ready to receive them and conversely that when knowledge reaches a certain stage a particular new idea is inevitable and is bound to occur to someone.

"We can now see that as structures become more and more complex and their analysis correspondingly involved the concept of successive approximations was certain to emerge and, as we all know now, Southwell and Cross were working simultaneously on essentially similar lines. Moment-distribution, which brought the esoteric mystery of rigid frame analysis within the competence of a good draughtsman, soon spread widely among structural engineers and the basic process and its many variants and developments are now part of the stock-in-trade of every design office.

"Southwell's relaxation process has proved to be the more general and has influenced thought in many branches of science. The discovery that a plate could be replaced by an analogous network, or in mathematical terms that finite difference equations, soluble by a step-by-step process, could be used instead of continuous differential equations for which no exact solution was known, has proved to be of paramount importance. The stresses in concrete and masonry dams, for example, have been properly explored for the first time in this way.

"But gradually it has been realised that even these brilliant innovations have their own special limitations. The continuous beam, even of many spans, presents no difficulty; variations in the second moment of area need no longer be individually treated, with great labour, now that standard solutions for distribution, carry-over and sway correction factors are available. But the portal type frame under lateral load still calls for a lot of arithmetic even though the principles of sway correction are now well understood. Nor can a method which has to be repeated *de novo* for every change in the load system ever become really universal.

"Arithmetic, too, is a chancy business, necessitating checks and counter checks, and so we find that the possibility of handing over all this work to machines is attracting attention. Already a programme has been developed in the Manchester Computing Laboratory for dealing with rigid frames and a big power station structure has been analysed on the digital computer in a matter of minutes. The creative process of design does not lend itself to programming, however, and it seems that at suitable stages in the progress of a design the machine will have to be halted so that the intelligence of the operator can be brought to bear on the situation.

"This really brings my review of our subject as far as the present day; we are evidently in the middle, or perhaps towards the end of one of those periods of intense activity on the theoretical side which I mentioned at the beginning of my address. Two different lines of further development can now be perceived in which we are not yet ready to take a decisive step forward, although this may come before long.

"On the one hand there is the research which indicates that the assumptions, on which all theoretical work so far is based, are useless and should be discarded, and on the other there is the development of new constructional forms for which acceptable methods of analysis have yet to be devised.

"May I spend a few more minutes in enlarging on these points?

"So far as bare frames are concerned the work of Professor Baker and his colleagues has opened new possibilities, although it is true that the appealing simplicity of his early work has become rather hidden as new complications have revealed themselves. The basic idea, that the post-elastic behaviour of structures is unconsciously relied upon by designers and should be considered from the outset, has stimulated a great deal of work on the theory of plasticity and on its applications to practice. Enthusiasts for this approach were able to point out that riveted joints were universally designed by limit design methods, but the extension of the concept to structures as a whole is a major step which has yet to be surmounted. It is clear, for example, that where deflections are a ruling consideration an elastic analysis can hardly be avoided. Then there are the problems of rolling and repeated loads which have hardly been touched as yet, and finally the all-important question of the load-factor is still quite open even in the simpler cases.

All these considerations may be side-stepped, so far as building frames are concerned, if the work of Dr. Thomas and his colleagues at the Building Research Station comes to fruition. They are showing that the clothing of a frame—panel walls and so on—make important contributions to its strength, much as a sheet of paper glued over a child's hoop transforms an easily distorted ring into quite a useful wheel. No one can say as yet where all this will lead to, but if it means that frames which have hitherto been split into individual units—floors, beams and columns—for convenience in design must really be treated as a whole, then quite clearly the analyst is in for a rough time.

"New forms of construction, however, offer the greatest challenge. There is already an extensive literature on shell roofs, to take one example, but the theory is still in an unsatisfactory state. On the one hand the exact mathematical analysis does not lend itself to the inverse process of design, and on the other experiment seems to show that the basic assumptions of that analysis are not fulfilled in practice.

"But when one learns that egg-shells weighing only 1/100 lb. have carried before failure endwise loads of 130 lb. distributed over about half the projected area one is forced to recognise that the shell or dome is a structural form of great importance.

"We stand, in fact, at the end of the two-dimensional age; space frames, shells and domes are bound to be

used more and more ; the engineer will have to get used to three-dimensional thinking, and no doubt this will have its repercussions on educational methods.

"It is unfortunate that architects, who are accustomed to thinking in three dimensions, are not usually so skilled in their understanding of the mechanics of structures, while engineers—chained as they nearly always are to that inexorably flat object, the drawing board—so often find that their imaginations will not leave the planes behind and soar away into space.

"Here then is the challenge which our profession and this Institution must first recognise and then face. New materials and processes and techniques are opening up all sorts of possibilities, but every new opportunity brings with it new problems for the designer. Structural design, no less than aircraft—or ship—or even textile design, is both an art and a science. On the scientific side there is little doubt that our analytical skill can keep pace with technical advances. But when we turn to the art of design—by which I mean the exercise of creative imagination based on a deep and intuitive understanding of structural principles—I am less confident. These are not easy ideas to put into words, but everyone here has experienced the sense of rightness with which one contemplates a great structure—the sense of inevitability, the certainty that in no other way could the designer have done the task which faced him. This is what one feels at the sight of the dome of St. Paul's rising above the roofs of London, of the Forth Bridge striding confidently from Lothian to Fife, of the Golden Gate bridge, or even—for this is not a question of size—when one enters a little Norman chapel, perfectly proportioned and a thousand years old.

"These are the visions we must not forget, however clouded they may be by building regulations—or by differential equations."

The President, 1954-1955—*continued*

Engineering. Since 1947, he has been an Examiner in Structural Engineering for the City and Guilds of London Institute.

Dr. Hamilton has contributed several papers to the Institution's Journal, including one on "Masonry Construction," in 1939, on "The Repair of Bomb-damaged Buildings" in 1945, and on "Old Cast Iron Structures" in 1949.

In the course of his thirty years' association with the Institution, Dr. Hamilton has worked untiringly to promote its aims and objects and his detailed knowledge of the work of its Committees will prove invaluable in directing its affairs as President during the coming Session. Among the large number of friends whom he has made during this period will be many who have pursued their technical studies under his guidance. His long experience in the educational field is a guarantee of his warm interest in the younger members of the profession. Members in all grades will wish him well in the coming Session and those who will be privileged to work under his leadership know that they may look forward to a year of continued progress for the Institution.

Institution Notices and Proceedings

PRESIDENTIAL ADDRESS—SESSION 1954-55

A General Meeting of the Institution of Structural Engineers will be held at 11, Upper Belgrave Street, London, S.W.1, on Thursday, October 7th, 1954, at 6 p.m., when Dr. S. B. Hamilton, Ph.D., M.Sc., B.Sc. (Eng.), M.I.C.E., M.I.Struct.E., will be installed as President for the Session 1954-1955, and will give the Presidential Address.

FORTHCOMING MEETINGS

Thursday, October 28th, 1954

Ordinary General Meeting for the election of members at 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when Mr. Frederick W. Slatter, M.I.Struct.E., M.Cons.E., M.Inst.W., M.Soc.C.E.(France), and Mr. Arthur Brown, A.M.I.Mech.E., M.Soc.C.E.(France), will give a paper on "Foundations, Underpinning and Structural Problems at the Daily News Building in the City of London."

Thursday, November 11th, 1954

Joint Meeting with the British Section of the Societe des Ingenieurs Civils de France at 6 p.m., when Monsieur N. Esquillan, M.Soc.C.E.(France), will give a paper on "Examples of Precast Ferro-Concrete Constructions in France."

Thursday, November 25th, 1954

Ordinary General Meeting, 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when Mr. D. R. R. Dick, B.Sc., M.I.C.E., will give a paper on "The Design and Construction of the Nuclear Reactor Buildings at Windscale Works, Sellafield."

Thursday, December 16th, 1954

Ordinary General Meeting, 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when Mr. M. F. Palmer, M.I.C.E., M.I.Struct.E., will give a paper on "Fabrication and Erection of Steel Plate Girder Railway Bridges."

Members wishing to bring guests to the Ordinary Meetings announced above are requested to apply to the Secretary for tickets of admission.

EXAMINATIONS—JANUARY, 1955

The Examinations of the Institution will next be held at centres in the United Kingdom and Overseas on January 11th and 12th, 1955 (Graduateship), and January 13th and 14th (Associate-Membership).

HONOURS AWARD

In offering their sincere congratulations to the following member on the distinction recently conferred upon him, the Council feel they are also expressing the good wishes of the Institution :—

ORDER OF ST. MICHAEL AND ST. GEORGE—C.M.G.
Mr. N. Wynne-Jones, C.B.E. (Member).

CERTIFICATE OF COMMENDATION

A Certificate of Commendation has been awarded to Mr. A. J. Harris, B.Sc.(Eng.), A.M.I.C.E., for a paper entitled "Hangars at London Airport—Design of Large Span Prestressed Concrete Beams."

RESEARCH SCHOLARSHIP

The Aluminium Development Association Research Scholarship in the use of light alloys in structural engineering for the year 1954 has been awarded by the Council of the Institution of Structural Engineers to Mr. E. M. Jubb, B.Sc.

MACLACHLAN LECTURE COMPETITION, 1955

The closing date for the receipt of entries for the next Maclachlan Lecture Competition is Thursday, March

31st, 1955. The general conditions of the competition are as follows :

1. The Institution of Structural Engineers shall institute a written lecture to be known as the Maclachlan Lecture and to be held annually.

2. The subject of the Lecture may be on any aspect of Structural Engineering as long as in every second year the subject shall be confined to steel structures. (This will be the case in 1955.)

3. Entrance into the competition for the Lecture shall be confined to Associate Members of the Institution, who are under the age of 32 years.

4. All papers entered for the competition shall be submitted to assessors to be appointed by the Council of the Institution, and all such papers (including the prize-winning Lecture) shall be available for publication in the Journal of the Institution at the discretion of the Council.

5. No paper submitted shall have been published or read elsewhere.

6. The winner of the competition shall be required to present the Lecture to a meeting of the Institution at which he will be presented with the sum of £17 10s.

7. Should a competitor's paper be considered worthy of ranking second in merit he will receive a consolation award of £5.

8. In the event of there being no winner of the competition in any one or more years, whether because no lecture submitted is considered to be of sufficient merit to warrant award, or for any other reason, the Institution shall transfer the above sums to the Research Fund of the Institution.

Particulars of the Competition for 1955

1. The Maclachlan Lecture will be given at a meeting of the Institution to be arranged towards the end of 1955.

2. The subject of the Lecture shall be confined to steel structures.

3. The work should be submitted as the script of a lecture which the author, if successful in the competition, will deliver before an audience in the course of about one hour. The development of mathematical formulæ and detailed calculations should be avoided as far as possible in the text ; if they are essential they should be embodied in appendices. Photographs, drawings, graphs, etc., which would appear as illustrations to the lecture in published form, should accompany the script. If additional illustrations would be shown as slides, a list of these should be included.

4. Six copies of each Lecture should be submitted and should be addressed to the Secretary of the Institution.

5. The closing date for the receipt of entries by the Institution is Thursday, March 31st, 1955.

PAPERS FOR PUBLICATION

The Literature Committee would be glad to consider offers of papers for presentation at the Institution or for publication in the Journal.

The following is a summary of the Committee's requirements relating to articles and papers : a copy of the full conditions may be obtained from the Secretary.

(1) Articles must be of an appropriate character, having a bearing upon structural engineering or upon some kindred scientific or constructional subject, and must be approved by the Literature Committee. A short title is an advantage.

(2) Contributions must be original either in subject-matter or in presentation. Articles which have already been published or have been read to other organised bodies, or are carelessly prepared, will not be accepted for publication.

(3) The style of writing will necessarily vary with the individual, but authors are requested to write as plainly and simply as their subject will allow. Papers should be written in the third person.

(4) Where the subject allows, a brief introduction or synopsis should state clearly the purpose and scope of the paper or article, and the author's conclusions or recommendations should be summarised at the end of the paper.

In order to facilitate the indexing of articles for reference, the author will be required in addition to prepare a short precis not exceeding 25 words for inclusion under the title of the paper on the contents page of the Journal.

(5) Illustrations are desirable where they assist in explaining the context or are fundamental to the subject. They should not be used if unnecessary for these purposes. Illustrations may be either line drawings or photographs.

(6) Line drawings must be specially prepared for reproduction on smooth white paper, or clear tracing paper, with heavy main lines and large clear lettering drawn in Indian ink with a mapping pen. Alternatively, the author may submit drawings on one sheet of paper with the relevant lettering on a cover sheet of tracing paper.

The printed page of THE STRUCTURAL ENGINEER is 7 in. wide by 10 in. deep. The drawings, where practicable, should be prepared not larger than twice this size with a view to half-scale reproduction. Unavoidably large drawings which require reduction to one-third size or less, must be specially heavy and with proportionately large lettering for clear reproduction. Ordinary working drawings are not satisfactory.

(7) Where photographs are submitted they should be printed *black on glossy paper*.

(8) MS. typewritten in double spacing should be submitted in duplicate.

Brevity is an advantage and papers should not normally exceed 7,500 words in length.

LECTURE COURSE ON DESIGN OF WELDED STRUCTURES

The lecture course on the Design of Welded Structures which, for some years past, has been held at the Brixton School of Building, will this year be held, by courtesy of the Institute of Welding, at the offices of the Institute at 2, Buckingham Palace Gardens, Buckingham Palace Road, S.W.1. Admission to the course is no longer restricted to graduates, but preference will still be given to graduates in the Theory of Structures, although the syllabus does not entail the analysis of complete frames. Most of the lectures deal mainly with the design and detailing of welded connections for various forms of welded steelwork, but two are devoted to metallurgical considerations, and the questions of fabrication erection and the estimating of welding quantities and costs are also included. In addition there are two practical lecture-demonstrations which are intended to give the students some first-hand knowledge of the arc welding process from the point of view of both accessibility and the standard of weld quality.

All lectures take place on Monday evenings from 6.30 to 8.30 p.m., commencing on October 4th, 1954, and the duration of the course is some 25 weeks. Applications to attend should be made to the Secretary, L.C.C. Brixton School of Building, Ferndale Road, London, S.W.4.

LONDON GRADUATES' AND STUDENTS' SECTION

A meeting of the Section will be held at 11, Upper Belgrave Street, London, S.W.1, on Tuesday, October

19th, at 6 p.m., when Mr. L. Scott White, O.B.E., M.I.C.E., M.I.Struct.E. (Past President), will read a paper on "Government Offices, Whitehall Gardens, the special problem of the re-siting of an historic building." This will be followed by a visit on Saturday, October 23rd, at 10 a.m., to the site of the work to inspect the historic building, together with the new construction now in progress.

A Meeting of the Section will be held at 11, Upper Belgrave Street, London, S.W.1, on Tuesday, November 23rd, 1954, at which members of the Section are invited to give short talks of 15-20 minutes' duration on various aspects of structural engineering. A prize will be awarded for the most meritorious contribution. Will those wishing to contribute please advise the Honorary Secretary, J. A. Pope, 53, Cranleigh Drive, Leigh-on-Sea, Essex ?

BRANCH NOTICES

LANCASHIRE AND CHESHIRE BRANCH

The following meetings have been arranged :—

Friday, October 15th, 1954

The inaugural meeting of the Session will be held at the Walker Art Gallery, Liverpool, at 7.15 p.m., when the Chairman's Address will be given by Mr. W. D. Blades, M.I.Struct.E. At the conclusion of the Address, the following films will be shown : "Soils and Foundations," and "Loch Sloy." The President and the Secretary of the Institution will attend the meeting.

Monday, November 29th, 1954

At the College of Technology, Manchester, at 6.30 p.m. Mr. S. M. Cooper, A.M.I.Struct.E., on "The Design Features and Collapse Investigation of the Tacoma Narrows Bridge" (with the film).

Tuesday, November 30th, 1954

The above meeting will be repeated in Liverpool.

Tuesday, December 7th, 1954

Trip to John Summers & Sons, Steelworks Shotton, by the Liverpool members of the Branch.

Joint Hon. Secretaries : A. S. Sinclair, A.M.I.Struct.E., 17, The Circuit, Cheadle Hulme, Cheshire ; M. D. Woods. A.M.I.Struct.E., 58, Spring Gardens, Salford.

MIDLAND COUNTIES BRANCH

The following meetings have been arranged :—

Saturday, October 9th, 1954

Annual Dinner, at the Botanical Gardens, Birmingham. The President and the Secretary of the Institution will be present.

Friday, October 22nd, 1954

At the James Watt Memorial Institute, Birmingham, 6 p.m. Chairman's Address by Mr. W. Phillips, M.Eng. (Sheffield), M.I.C.E., M.I.Struct.E. The President and the Secretary of the Institution will be present.

Thursday, November 4th, 1954

At the Public Library, Stafford, 7 p.m. Mr. Edgar Morton, M.Sc., P.A.Inst.W.E., Hon.M.I.Q., on "Examples of Site Exploration."

Tuesday, November 16th, 1954

At the Supper Room, The King's Hall, Queen Street, Derby, 7 p.m. Mr. E. Williams on "The Construction of a Prestressed Concrete Gas Holder Tank at Cromer."

Friday, November 26th, 1954

Joint Meeting with the Birmingham and Five Counties Architectural Association, at the James Watt Memorial Institute, Birmingham, 6 p.m. Mr. S. Woolf, on "Structural Use of Timber."

Hon. Secretary : L. A. Firminger, A.M.I.Struct.E., 656, Chester Road, Erdington, Birmingham, 23.

GRADUATES' AND STUDENTS' SECTION

The following meetings have been arranged :—

Friday, October 29th, 1954

Joint Meeting with the Graduates' and Students' Section of The Institution of Civil Engineers. Mr. R. D. Mackey, B.Sc.(Hons.), A.M.I.C.E. (Graduate), on "Site Investigation for Foundations." At the Birmingham Civic Centre, 6 p.m.

Tuesday, November 30th, 1954

Details to be announced.

Hon. Secretary : H. L. Bramwell, 139, Wood Lane, Handsworth Wood, Birmingham, 20.

NORTHERN COUNTIES BRANCH

The following meetings have been arranged :—

Tuesday, October 12th, 1954

At Middlesbrough. Chairman's Address by Mr. W. G. Gentry, M.I.Struct.E., A.M.I.C.E. "Some Reflections on a Designer's Progress." The President and the Secretary of the Institution will be present.

Tuesday, November 2nd, 1954

At Middlesbrough. Mr. G. Little, M.Sc., on "Further Studies of Load Distribution in Bridge Decks."

Wednesday, November 3rd, 1954

The above meeting will be repeated at Newcastle.

Wednesday, December 1st, 1954

At Newcastle. Mr. A. P. Clarke, B.Sc., on "Lackenby Steelworks."

Tuesday, December 7th, 1954

At Middlesbrough. Mr. E. Czeiler, M.I.Struct.E., on "Steelworks for Hindhaugh Street Flats, Newcastle."

All meetings commence at 6.30 p.m., preceded by buffet tea at 6 p.m., the Tees Centre Meetings being held in the Cleveland Scientific and Technical Institution, Corporation Road, Middlesbrough, and the Tyne Centre Meetings in the Neville Hall, Westgate Road, Newcastle upon Tyne.

Hon. Secretary : Capt. O. Lithgow, A.M.I.Struct.E., 4, Stoneleigh Avenue, Acklam, Middlesbrough.

NORTHERN IRELAND BRANCH

The following meetings have been arranged :—

Tuesday, October 5th, 1954

Opening Meeting of the Session, at the College of Technology, Belfast, 6.45 p.m., preceded by tea at the Overseas League premises, Wellington Place, Belfast, at 6 p.m. Mr. J. Singleton-Green, M.Sc., M.I.C.E., M.I.Struct.E., A.M.I.Mech.E. (Member of Council), on "Concrete as an Engineering Material."

Tuesday, November 2nd, 1954

Annual Dinner and Social Function, at the Grand Central Hotel, Belfast, 6.30 p.m. Visit of the President and the Secretary of the Institution.

Tuesday, December 7th, 1954

At the College of Technology, Belfast, 6.45 p.m. Mr. W. S. Atkins, B.Sc., M.I.C.E., M.Inst.W., on "Structural Steelwork and Concrete Construction, with particular reference to Abbey Steelworks."

Hon. Secretary : A. H. K. Roberts, B.A., B.A.I., M.I.Struct.E., M.I.C.E.I., "Barbizon," 26, Dunlambert Park, Belfast.

SCOTTISH BRANCH

The following meetings have been arranged :—

Monday, October 25th, 1954

At the Ca'doro Restaurant, Glasgow, 6 p.m. Chairman's Address by Mr. W. Heigh, M.I.Struct.E. Visit of the President and the Secretary of the Institution.

Tuesday, October 26th, 1954

Annual Dinner and Dance, at the Grosvenor Restaurant, Glasgow. The President and the Secretary of the Institution will be present.

Wednesday, November 17th, 1954

Joint Meeting with the West of Scotland Branch of the Institute of Welding, at the Institution of Engineers and Shipbuilders, 39, Elmbank Crescent, Glasgow, 7 p.m. Mr. S. M. Reisser, B.Sc.(Eng.), M.I.Struct.E., A.M.I.C.E., on "The Influence of Welding on Steel Building Structures."

Friday, December 3rd, 1954

Joint Meeting with the East of Scotland Branches of the Institute of Welding and the Society of Engineers. Paper on Welded Structures. At the Heriot Watt College, Chambers Street, Edinburgh.

Hon. Secretary : G. Drysdale, A.M.I.Struct.E., "Niaroo," 33, Union Street, Motherwell, Lanarkshire.

SOUTH-WESTERN COUNTIES BRANCH

The opening meeting of the Session will be held at the Duke of Cornwall Hotel, Plymouth, on Saturday, November 13th, at 6.30 p.m., when the Chairman's Address will be given by Lt.-Colonel R. Hazzledine, O.B.E., T.D., M.I.Struct.E. The President and the Secretary of the Institution will attend the meeting, which will be followed by an informal dinner.

Joint Hon. Secretaries : E. W. Howells, M.I.Struct.E., 10-12, Market Street, Torquay; C. J. Woodrow, "Elstow," Hartley Park Villas, Tavistock Road, Plymouth.

WALES AND MONMOUTHSHIRE BRANCH

The following meetings have been arranged :—

Tuesday, October 19th, 1954

At Cardiff. Chairman's Address, by Mr. F. V. M. Bell, M.Eng., M.I.C.E., M.I.Struct.E. Visit of the President and the Secretary of the Insitution.

Tuesday, November 2nd, 1954

At Swansea. The Chairman's Address will be repeated and will be followed by a discussion.

Saturday, November 6th, 1954

At Colwyn Bay. The Chairman's Address will be repeated and will be followed by a discussion.

Tuesday, November 16th, 1954

At Cardiff. Joint Meeting with the Institute of Welding. Mr. F. Brooksbank, M.A.(Cantab.) (Graduate) on "Economics in Welding Design."

Wednesday, December 8th, 1954

At Swansea. Mr. H. E. Lewis, B.Sc., D.I.C. (Graduate) on "Developments in the Structural Use of Concrete."

Meetings at Cardiff will be held at the South Wales Institute of Engineers, Park Place, at 6.30 p.m.

Meetings at Swansea will be held at the Mackworth Hotel at 6.30 p.m.

Meetings at Colwyn Bay will be held at the County Buildings at 6 p.m.

Hon. Secretary : K. J. Stewart, A.M.I.C.E., A.M.I.Struct.E., 15, Glanmor Road, Swansea.

WESTERN COUNTIES BRANCH

The following meetings have been arranged :—

Wednesday, October 20th, 1954

Chairman's Address, by Mr. E. N. Underwood, B.Sc.(Eng.), M.I.C.E., M.I.Struct.E. Visit of the President and the Secretary of the Institution.

Friday, November 12th, 1954

Combined Meeting with the Institution of Civil Engineers. Mr. H. C. Husband, B.Eng., M.I.C.E., M.I.Struct.E., M.I.Mech.E. (Member of Council), on "Unusual Industrial Structures."

Friday, December 3rd, 1954

Mr. J. Guthrie Brown, M.I.C.E., M.I.Struct.E. (Vice-President), on "Highlights in an Engineer's Life."

Friday, December 10th, 1954

Combined Dance, Royal Hotel, Bristol.

Unless otherwise stated, all meetings will be held in the University of Bristol Geology Lecture Theatre (entrance, University Road), at 6 p.m., preceded by tea at 5.30 p.m.

Hon. Secretary : E. Hughes, A.M.I.Struct.E., 23, Southdown Road, Westbury-on-Trym, Bristol, 9.

YORKSHIRE BRANCH

The following meetings have been arranged :—

Wednesday, October 13th, 1954

Chairman's Address, by Mr. Leslie Preston, M.I.Struct.E. Visit of the President and the Secretary of the Institution.

Wednesday, November 17th, 1954

Mr. D. V. Pike, M.I.Struct.E., A.M.I.C.E., on "Aluminium Alloy Structures."

Wednesday, December 15th, 1954

Mr. A. P. Clark and Mr. T. V. Thompson, M.I.Struct.E. on "Lackenby Steelworks."

All meetings will be held at the Great Northern Hotel, Leeds, at 6.30 p.m.

Hon. Secretary : E. Wrigley, A.M.I.Struct.E., 17, The Drive, Alwoodley, Leeds.

UNION OF SOUTH AFRICA BRANCH

Branch Hon. Secretary : A. E. Tait, B.Sc., A.M.I.C.E., A.M.I.Struct.E., P.O. Box 3306, Johannesburg. During week-days Mr. Tait can be contacted in the City Engineer's Department, Town Hall, Johannesburg. 'Phone : 34-1111, Ext. 257.

Natal Section Hon. Secretary : E. G. Bennett, A.M.I.Struct.E., c/o The Reinforcing Steel Co., Ltd., P.O. Box 49, Merebank, Durban.

Cape Section Hon. Secretary : R. Stubbs, M.I.Struct.E., The Reinforcing Steel Co., Ltd., P.O. Box 2692, Cape Town.

The Design and Construction of the Nuclear Reactor Buildings at Windscale Works, Sellafield*

By D. R. R. Dick, B.Sc., M.I.C.E.

Synopsis

A general description is given of the location, lay-out and function of Windscale Works, with a resumé of the fundamental structural requirements associated with the Nuclear Reactors.

The design of the foundations for the Reactors and the enclosing buildings was conditioned by the glacial moraine forming the sub-soil, and the measures taken to allow for differential settlement are described.

The work above ground level consisted broadly of the steel-framed buildings, the concrete shield to the Reactors, and the ventilation shafts, the design and construction of each section being covered in fair detail. The main features considered, are the effect of deflections on the design of the portal-framed buildings, the use of Bailey Bridge girders to support the shuttering for the concrete shield over the Reactors, and the introduction of the filter galleries at a late stage in the construction of the ventilation shafts.

In conclusion, a few brief comments are given on the construction programme and the difficulties facing the engineers and contractors in carrying out work in this new field.

Introduction

In 1946 the Ministry of Supply, in association with the Ministry of Works, were given instructions to proceed with the design and construction of the Harwell Atomic Energy Research Establishment and the factories required for the production of fissile material.

By June, 1947, the work at Harwell and at Springfields—where the uranium ore is processed for use in Reactors—was sufficiently far advanced for attention to be given to the second stage in the programme, namely the construction of two large Reactors for the production of plutonium.

The site chosen for this project lies on the Cumberland coast at Sellafield, and was at that time occupied by a disused Ordnance factory in course of demolition. Many buildings remained, however, particularly in the Administration Group, and their existence, in addition to the roads and services, contributed largely towards a speedy start being made on the construction of the atomic factory now known as Windscale Works.

Sub-soil Investigation

Previous experience of the area indicated that the sub-soil would consist of a glacial drift overlying red sandstone, and that the sandstone would be found at depths too great to be used as a bearing stratum for the Reactor foundations. In view, however, of the known presence of faults in the sandstone it was important to establish its level.

A series of bore-holes was therefore put down, which established the rock level on the site of the Reactors at a depth of 70 to 100 feet. These bore-holes also revealed the presence of clay lenses up to 4 feet thick, without, however, revealing their extent.

As it was intended to locate the underside of the Reactor foundations about 15 feet below ground level, it was decided to sink a further series of bore-holes to a depth of 40 feet at the proposed siting position, and to adjust the siting of the Reactors to give the most favourable conditions. From the results obtained, it was found possible to site the Reactors so that no major clay lenses occurred in the 10 feet immediately under the foundations.

The glacial drift consisted of sand, gravel and boulders in a matrix of silt and clay, and its bearing capacity was assessed at an average value of $2\frac{1}{2}$ tons per sq. ft. with a peak pressure of 3 tons per sq. ft. These figures were largely determined by the necessity to restrict settlement to a low value, and it was for this reason that great care was taken to remove any clay lenses at foundation level, and to site the Reactors clear of all major lenses.

When the foundation rafts were completed, level plugs were set in the top surface and readings taken continuously throughout the construction period. The maximum settlement recorded over the two years was $\frac{5}{8}$ in. and was uniform over the area covered by each raft.

Layout of Reactor Group

The Reactor Group consists of two Reactors centrally disposed about a large storage pond, where the uranium cartridges cool down after being irradiated in the Reactors. These cartridges, on being discharged from the Reactors, fall into a water duct and are transported by means of an under-water railway to the storage pond. Here they are stripped of their aluminium cans, placed in lead containers, and conveyed to the chemical separation plant for processing.

Each Reactor is housed within an asbestos cement-sheeted building, 200 feet long by 90 feet wide and 140 feet high, centrally disposed between the two blower houses from which cooling air is supplied to the Reactor (Fig. 1). These blower houses act as intake chambers for the fans and contain galleries in which both wet and dry filters are located. The external cladding therefore consists largely of aluminium louvres, with brick in-filling, particularly on the gables.

The cooling air passes through reinforced concrete air ducts to the Reactor and thence up the 412 feet high ventilation shafts at the top of which it is filtered once more. These filters may contain active particles and therefore have to be placed in lead containers before they can be handled for disposal. An external lift shaft is therefore attached to the ventilation shaft for the conveyance of these lead containers to and from the filter gallery.

The Nuclear Reactors

A schematic diagram of the Windscale Reactors is given in Fig. 2, from which it will be seen that they

Paper to be read before the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1, on Thursday, November 25th, 1954, at 6 p.m.

consist of a large mass of graphite in which are located numerous horizontal channels containing the uranium cartridges. This mass of graphite is built up from individual blocks machined to fine tolerances so that each channel is accurately positioned within ± 0.015 in. At right-angles to the uranium channels, in both horizontal and vertical planes, additional channels are formed in which the control and shut-down mechanism operates.

Surrounding the Reactor is a reinforced concrete biological shield up to 9 feet thick, which is lined internally with aluminium sheet, bolted back to the concrete. Between the concrete face and the graphite is interposed a layer of 6-inch steel plate known as the thermal shield. These plates slow down the thermal neutrons and thereby absorb much of their energy as heat and in this way prevent a similar effect developing in the concrete biological shield. To prevent the heat absorbed by the steel plates being radiated to the concrete, the plates are backed with aluminium boxes filled with insulating material, and an air passage is provided between these boxes and the aluminium sheeting on the concrete face.

The fans in the blower houses deliver air under slight pressure to the face of the graphite via the air ducts on either side of the Reactor. Thence the air passes along the uranium channels to the outlet ducts leading to the ventilation shaft. Air for cooling the back of the insulation boxes is taken off at the graphite face and carried in metal ducts housed within the biological shield to appropriate points on all faces.

The design and construction of the Reactors therefore involved the following fundamental requirements :—

(1) The concrete biological shield must be air-tight and of a minimum density at all points of 145 lb. per cubic foot, and must be designed to take account of all temperature effects.

(2) All control and charging arrangements for the Reactor are operated from outside the biological shield, and enter the Reactor through tubes which must be set accurately in the concrete work so as to line through with the channels in the graphite.

(3) The thermal shield plates must be allowed to expand under temperature effects without masking any of the tubes through the biological shield.

Fig. 1.—General view of Reactor Group [*Crown Copyright Reserved*

(4) The steel base plates supporting the graphite must be level to an accuracy of ± 0.015 in.

(5) The water duct must be completely sealed against seepage into the ground, as the water might be contaminated by radio-active particles.

There are a number of other requirements associated with radiation hazards, but it is not proposed to go into these aspects of the design in this Paper.

Foundations

It will be seen from Fig. 3 that the foundations consist of a 10 feet thick raft carrying the Reactor and the ventilation shaft, with independent foundations for the steel-framed buildings surrounding the Reactor. The load transmitted to the sub-soil by this raft exceeds 57,000 tons, and there was therefore a desire to separate the water duct and air ducts from this raft by means of a

positive joint, which would allow for differential settlement.

The difficulty, however, of forming a joint in the air ducts close to the Reactor, which would not at the same time lead to a reduction in the efficacy of the concrete walls as a shield against radio-activity, will be appreciated ; it was therefore decided to form the joint remote from the Reactor and to design the intervening section of air duct for an upward pressure from the ground.

The water duct, however, could not be treated on similar lines owing to its lack of inherent strength. Furthermore, the formation of a joint which would be water-tight at all times under an appreciable head of water, was obviously not going to be easy. It was therefore decided to omit a section of the duct adjacent to the raft, and to construct this section at a late stage in the programme.

In the initial design the centre line of the ventilation shaft was fixed relative to the Reactor so as to achieve a uniform distribution of load on the sub-soil. At that stage the centre of loading lay within six inches of the centre of area of the raft, whose dimensions were 196 ft. by 100 ft. by 10 feet thick. As the design was developed it was not possible to preserve this close agreement and the final eccentricity was 3 feet, giving ground pressures of 3.03 and 2.50 tons per sq. ft.

So as to keep the raft thickness down to a minimum, it was designed as a flange to the superstructure, and the construction programme was carefully planned to allow for this fact. Nevertheless the reinforcement required in the raft amounted to 570 tons, the main bars being $1\frac{3}{8}$ in. diameter.

For its construction a batching plant was set up at one end delivering into skips on narrow gauge track and thence to two 3-ton Scotch derricks. Construction was carried out in 2 ft. 6 in. lifts and, to avoid placing reliance on the preparation of horizontal surfaces, a positive key was provided by upstanding ribs as shown in Fig. 4. These ribs were permanently shuttered in expanded metal and cast integral with the lift below.

The excavation for the first foundation raft and its construction took place over a period of five months from October, 1947, and during the same period all work on the foundations for the steel-framed buildings was also completed.

The main stanchion foundations for the Reactor building housed $2\frac{1}{2}$ in. dia. holding down bolts 10 ft. 9 in. long, which were surrounded by 5-in. dia. steel tubes held accurately in position by a light steel frame within the concrete. On the top surface of these foundations, and also of the raft, were constructed reinforced concrete jacking blocks to facilitate the positioning of the portal frame bases.

The stanchion foundations were interconnected with reinforced concrete cill beams carrying the dado walls and sheeting posts ; these cill beams were also designed to distribute the horizontal thrusts from the portal frames as between the raft and the foundations to the gable framing.

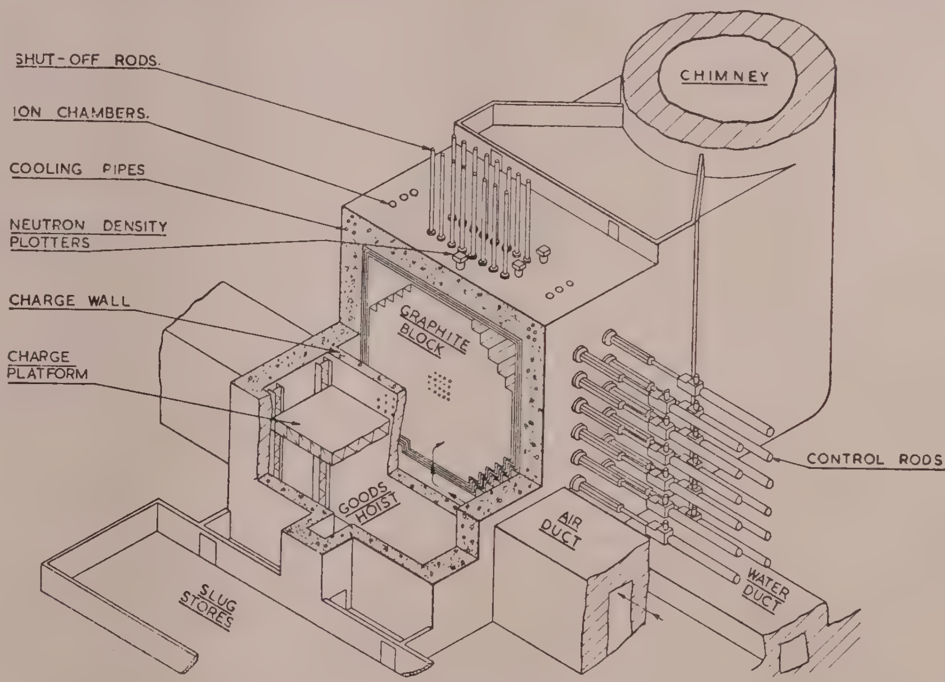

Fig. 2.—Isometric diagram of reactor

Design of Reactor Building

The fundamental requirement in connection with the Reactor building was the provision of an overhead crane gantry capable of lifting loads of up to 25 tons, and of placing them accurately in position. The decision to provide external cladding and thus form a building out of the gantry was determined by the following considerations :—

(1) By providing overhead cover during construction it was felt that the weather conditions would have less effect on the overall construction programme.

(2) The provision of all-weather equipment would have added to the design and supply problems, which were already numerous enough.

(3) It was not possible to assess very accurately the extent and nature of the operations that would have to be performed adjacent to the Reactor once it was in production.

Quite obviously the design of a single aisle portal frame building 140 feet high is conditioned by the permissible deflections, and the bases to the frames were therefore fixed at foundation level, while the tops of the frames

were propped by girders in the plane of the roof, connecting to a vertical bracing system in the gables.

The deflections that could be permitted were governed by the conditions at different stages in construction, namely :-

(a) when providing shelter during the construction of the Reactor.

(b) when, as a completed building, greater rigidity was required in connection with the accurate positioning of loads by the overhead crane, and when the four centre

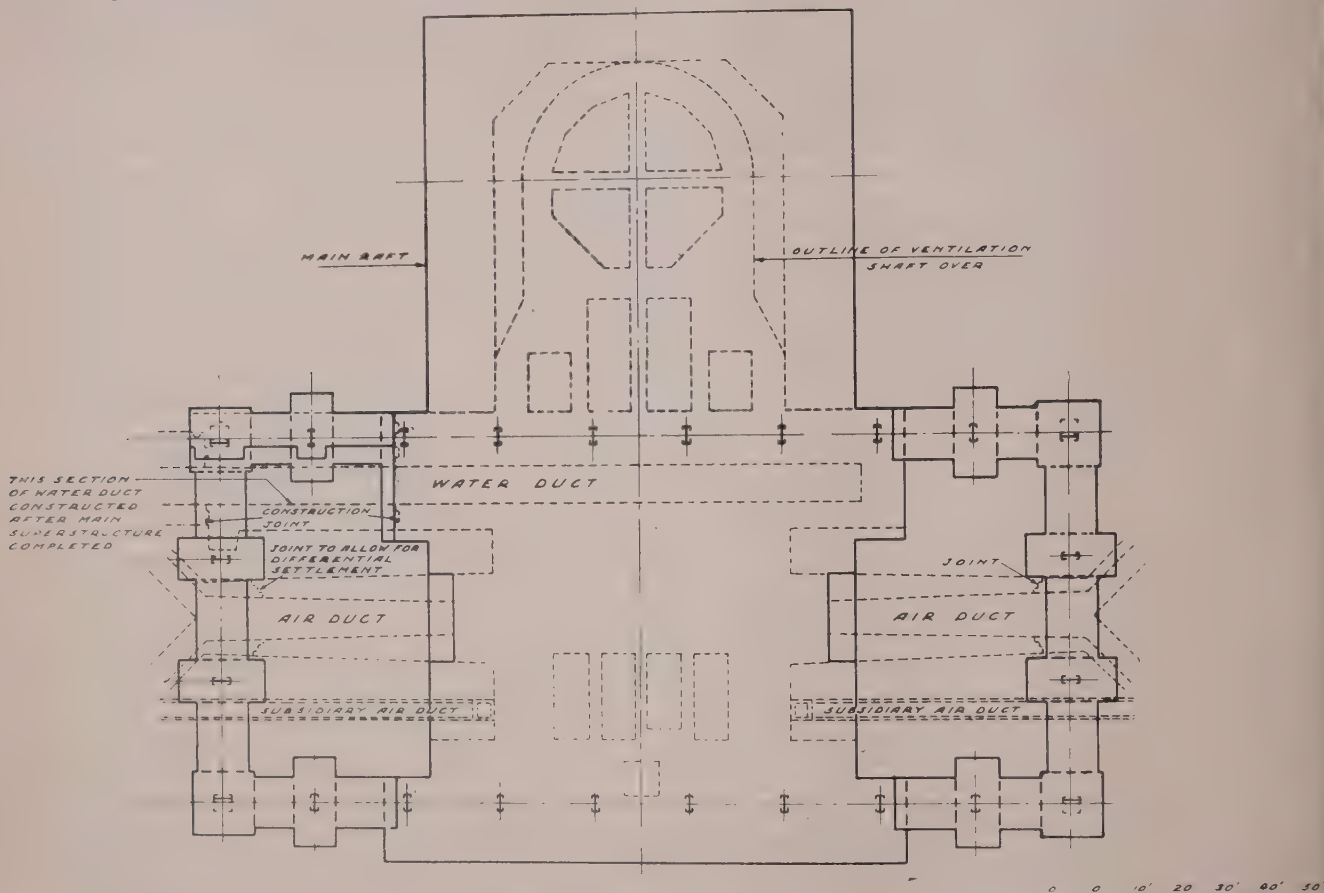

Fig. 3.—Plan of foundations

frames would be built into the concrete around the Reactor for a height of 80 feet above ground level.

Whereas the gauge widening at crane level was restricted to ¾ in. at both stages, the overall deflections during the construction period were only critical in so far as they might lead to damage to sheeting and glazing. A general picture of the calculated deflections and moments during this period can be obtained from Figs. 5 and 6, the forces due to wind being based on a mean wind velocity of 85 m.p.h. When the centre frames had been built into the concrete shield these deflections were substantially reduced and in no case were they more than half those in the initial stage.

Details of the portal frames are shown in Fig. 7 and 8, from which it will be seen that a riveted design was adopted, with a limited amount of shop welding where it would suit the fabricator's convenience. The frames are tied at vertical intervals of 30 feet by means of horizontal wind girts of all-welded lattice construction, from which the sheeting posts are stayed at each level. These sheeting posts occur midway between the main frames which are at 22-feet centres, the span of the sheeting rails being thereby limited to 11 feet.

The roof, which is of steel deck construction, is supported on 6 in. × 3 in. joist purlins spanning 22 feet, the deflection being reduced by means of outriggers from the bottom flange of the main roof members ; these outriggers in turn assist in stabilising these bottom flanges.

The vertical cladding is asbestos cement sheets, with horizontal bands of patent glazing. In view of the severe wind conditions the sheets are held to the rails with twice the normal number of hook bolts, and the glazing bars have been specially stiffened.

Although a runway was provided at eaves level from which cradles could be slung when cleaning windows or

Fig. 4.—Foundation raft during construction

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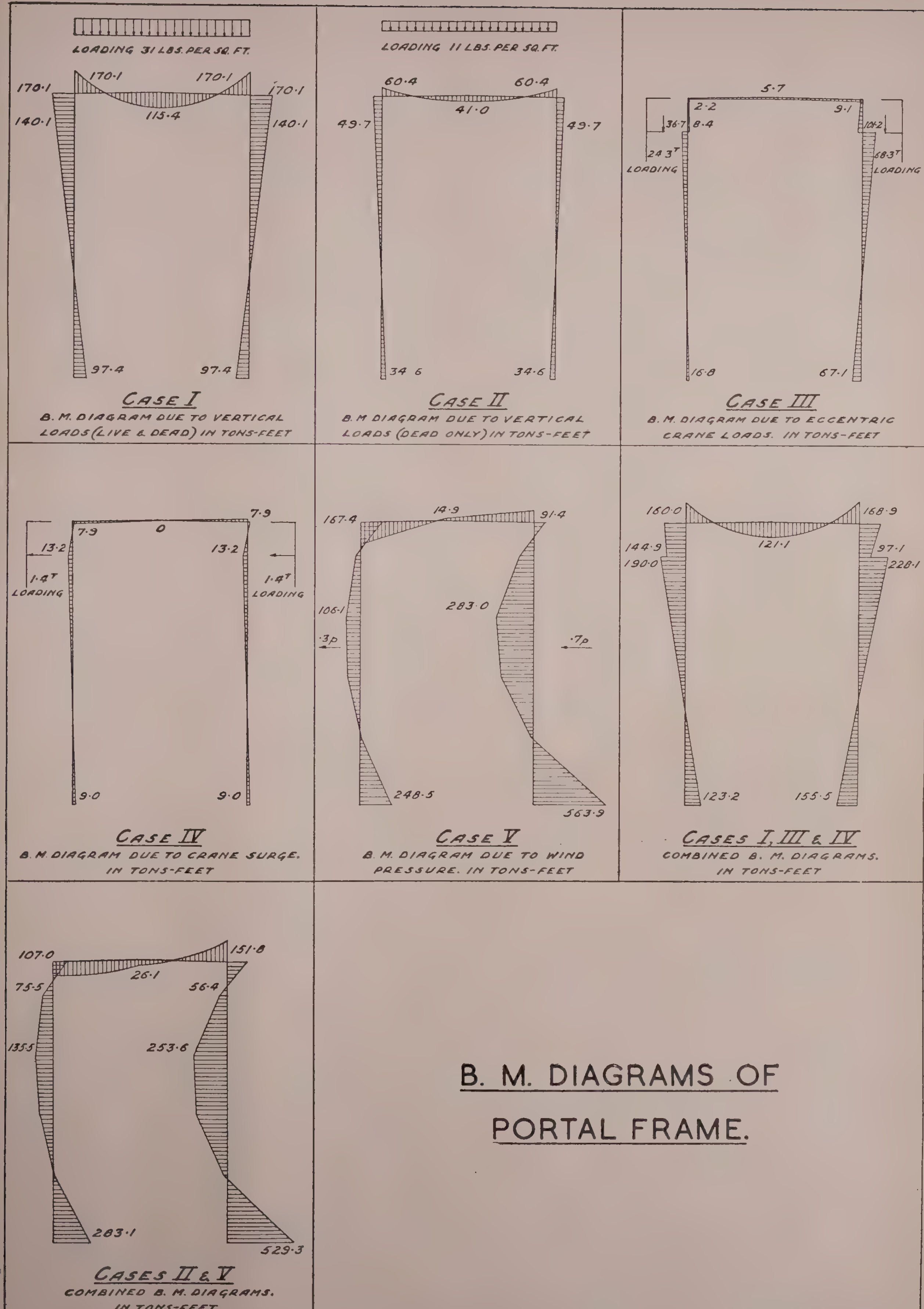

Fig. 5.—Moments on portal frame

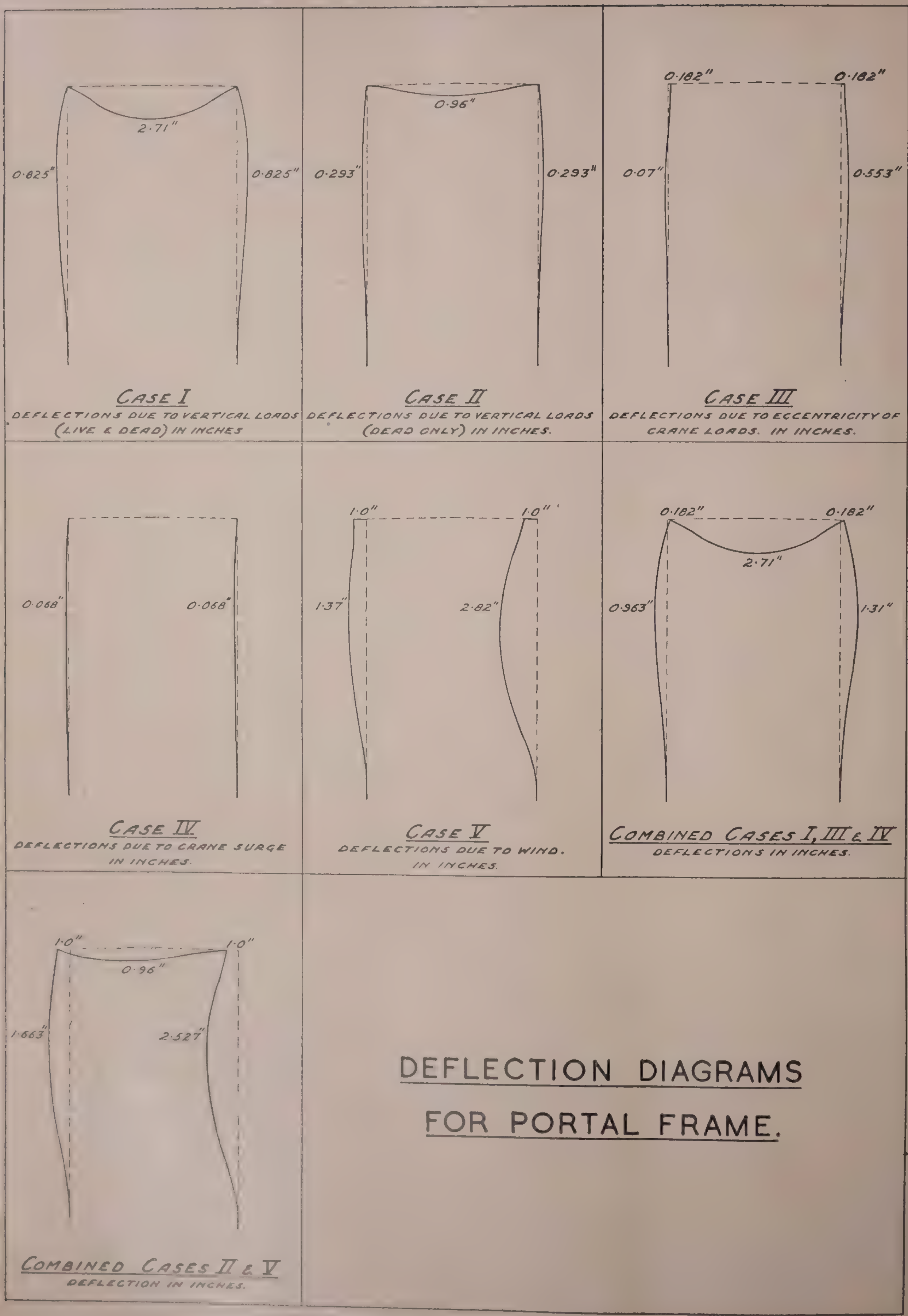

Fig. 6.—Deflection of portal frame

replacing glass and sheets, considerable expense is involved in carrying out these operations, and overall economies could have been effected by the use of a more robust type of sheeting, and a design of glazing bar—had such been available—which permitted the replacement of glass from inside the building.

Erection of Steel Framework

By virtue of the construction work on the concrete shield to the Reactor, it was not possible to erect the steel framework from one end, and two cranes working outwards from the centre bays had to be employed.

With a general level of excavation of 5 feet below ground level, the height to the apex of the portal frame was 145 feet, and the two 10-ton Scotch derricks were therefore mounted on gabberts 40 feet high, as can be seen in Fig. 9. These gabberts were in turn mounted on bogies running on standard gauge track, and were winched back as the erection progressed into the area finally occupied by the blower houses. This was possible owing to the excavated level, in this case also, being 5 feet below ground level so as to accommodate a multiplicity of ducts.

The gabberts were specially designed for the work by the steelwork contractors, and are of particular interest in that the vertical members consist of 6-ft. dia. steel cylinders, which were filled with ballast after erection, thus providing the necessary kentledge. Shutters near the base permitted the removal of the ballast prior to the gabberts being dismantled.

Although the 10-ton derricks were able to erect most of the steelwork, the bottom lengths of the portal legs weighed 14 tons each and these were handled by a mobile crane. Once in position they were anchored to the foundation by the 2½-in. dia. holding-down bolts, which were in turn connected to yokes passing over the base channels. Thereafter erection proceeded in such a way as to ensure that the tops of the centre two portals were braced to ground in a longitudinal direction before the roof members were hoisted into position.

Concrete Shield to Reactor

Mention has already been made of the fundamental requirements for this shield, and also of the principle adopted in the design of the raft, which envisaged careful planning of the construction of the superstructure. These requirements were met in practice by the following measures :—

(1) Uniform High Density

(a) All materials were carefully checked by the site laboratory, and aggregates were delivered and weighed in four separate sizes to ensure uniformity of mix.

(b) All aggregates were selected so as to give a dense concrete, a specific gravity of 2.89 being achieved with a quartz dolerite.

(c) All concrete was centrally weigh-batched, and was mixed in continuous drum mixers, from which it was delivered to the hoppers of the concrete pumps.

(d) The water/cement ratio was kept as low as practicable to minimise shrinkage, and a wetting agent was added to the mix to give sufficient workability for the concrete to be pumped.

(e) Concrete was placed in lifts not exceeding 3 feet in depth, and was compacted with immersion vibrators.

(2) Joints and the Prevention of Cracks

(a) The preparation of construction joints was carefully specified, and their position was located on the drawings so that they were staggered as between lift and lift, and also in plan, and so that no bay cast exceeded 25 feet in length.

(b) Alternate bay construction was adopted so that adjoining bays in the same lift were not cast without an interval of at least three days.

(c) A minimum of 0.1 per cent. shrinkage steel was provided in each face in the horizontal directions, and half of this in a vertical direction.

(3) Planning of Construction

The shield was divided for convenience of planning into three sections, the ventilation shaft base with its associated exhaust ducts, the Reactor walls including the inlet air and water ducts, and the charging section, and it was calculated that the maximum amount by which the other two sections could be allowed to get out of step with the Reactor section was 16 feet, i.e., the walls of the other sections must not at any stage be higher or lower than the Reactor section by more than 16 feet.

(4) Setting Out

(a) Centre lines were projected to fixed datum points well clear of the work, and within the area bounded by the concrete shield these centre lines were scribed on datum plates set in the concrete, these plates also being used as datum points for levelling.

(b) Where tubes passed through the walls they were supported on light steel frames, which were packed up from the lift of concrete below and bolted down before being concreted in. Care was taken in the design of these frames to avoid the introduction of features which would lead to air being trapped during concreting.

(c) One complete wall which was penetrated by hundreds of tubes at close centres was shuttered in steel plates made up into units complete with the tubes welded between them. Each unit was 20 feet long, 4 ft. wide and 3 ft. high, and was carefully levelled and positioned by reference to the unit already placed below it.

(d) Fixings for thermal shield guides, etc., were made of steel plates to which were welded tubes housing bolts, this detail affording limited play on the bolts.

(e) The base plates supporting the graphite were levelled off grillage joists set in the concrete, each plate having four set screws bearing on hardened steel packings welded to the top flange of the grillage joists.

(5) Temperature Effects

(a) Allowance was made in the design of the concrete shield for temperature gradients, expansion of the walls, etc., which gave rise to moments of the order of 5×10^6 lb. in. per ft. width.

(b) The thermal shield on the hot faces was housed in guides formed with broad-flanged beams, which in turn were held back to the concrete face by connections which permitted relative movement between the guides and the concrete.

(c) Aluminium boxes filled with glass wool blankets were fixed to the guides, and thus provided insulation behind the thermal shield. These boxes were so designed as to provide a reasonably effective air seal between the Reactor and the cooling air passage behind the boxes.

(d) The face of the concrete itself was covered with Alclad sheets fixed with expanding bolts penetrating the concrete to a depth of 5 in. The possibility of disintegration of the concrete face under neutron bombardment had to be envisaged and these sheets would prevent loose concrete blocking the air passages. The sheets were also used to canalise the air in these passages by turning the

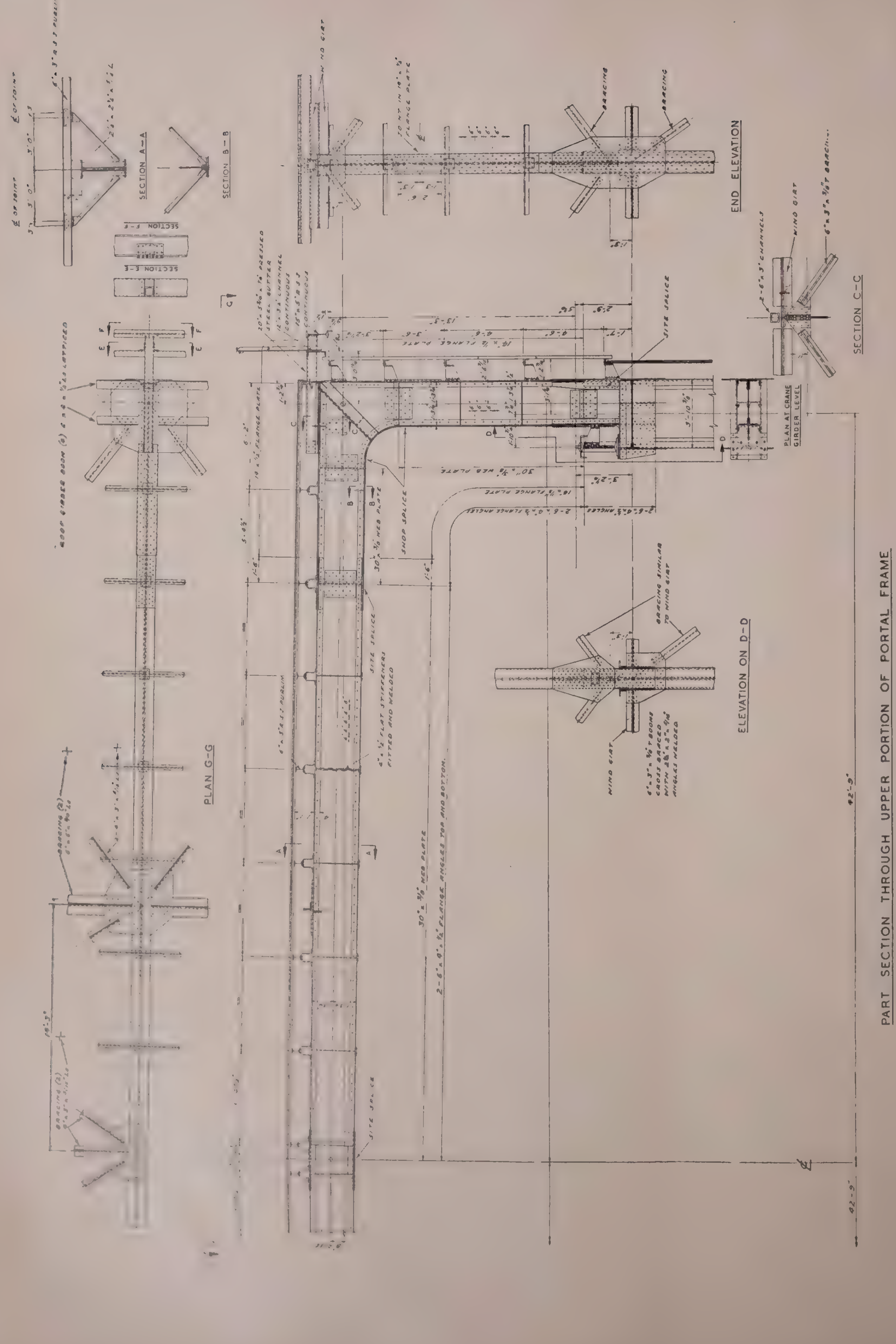

Fig. 7.—Details of portal frame steelwork

bottom of each sheet outward against the back of the insulation boxes.

In addition to the above measures, the tolerances specified on the accuracy of concrete faces was $\pm\frac{1}{2}$ in. relative to their true plan position at any point, with a maximum rate of divergence of $\frac{1}{4}$ in. in 10 feet. Bearing in mind the scale of the work, the achievement of this degree of accuracy was not an easy matter. In spite of this, the tolerances were rarely exceeded even though the transfer of datum points to all levels was extremely complicated due to the presence of scaffolding, plant, etc., and the solid barrier formed by intervening shield walls.

The quality of the work was in general well up to that specified, and when it fell short of the requirements this was due to one of the following causes :—

(a) The use of pumps meant that the " man on the spot " did not always have sufficient control of the rate of delivery of concrete in the more complicated sections of the work.

(b) The design of steel frames, tubes, etc., set in the concrete did not always preclude the possibility of air being trapped during concreting.

(c) Even though special aggregates were used and teams were trained on mock-ups, it was not found possible to place concrete to the specified density in the wall made up of steel plate shutters housing the hundreds of tubes at close centres. Any deficiencies were made good quite easily by pressure grouting.

(d) The rigidity of the steel shutters used was hardly sufficient for work to these exacting tolerances. But it is to be remembered that shuttering was in very short supply at that time.

Work on the construction of the shield walls started in the ventilation shaft section while structural steelwork was being erected for the portal frame building over the remaining sections, and therafter this section of the concrete shield was always in advance of the remainder. In all 20,800 cubic yards of concrete was placed in the walls of each Reactor, and the great majority of it was delivered to the concreting bays by pumps, which, in the top lifts, had to raise the concrete over 80 feet above ground level. Work was completed to the level at which the concrete roof was to be constructed over the Reactor in a period of six months, which meant that by the end of 1948 the main buildings were weathertight and the concrete work had reached a stage where a serious start could be made on the installation of the plant associated with the Reactor. As the initial excavation work had started in October, 1947, it had therefore taken 15 months to reach this stage.

Design and Construction of Concrete Roof over Reactor

The construction of the concrete biological shield roof over the Reactor presented several problems, the most important being to decide on the methods to be adopted to support the shuttering during the pouring of the concrete.

Several factors prevented the use of the normal methods of supporting from the underside, the principal ones being that :—

(a) The underside of the concrete slab had to be protected from the effects of the high temperature by a layer of mild steel plating 6 in. thick which had to be supported from the roof.

(b) Further protection in the form of a layer of insulation in metal boxes was laid on the top of the steel plating.

(c) A space between the top of these insulation boxes and the underside of the slab had to be provided for the passage of cooling air.

(d) Work inside the vault was to proceed simultaneously with the construction of the roof and the work was to be carried out under conditions of extreme cleanliness.

Thus, a method of suspending the shuttering, thermal shielding and insulation became the obvious solution to the problem and the choice was narrowed down to two main systems. The first was from a system of girders spanning across the Reactor walls and contained within the depth of the roof slab, and the second from a similar girder system placed above the slab.

The large number of holes which had to be formed in the slab placed limitations on the positions of the girders and it was decided to adopt the second method in order that advantage could be taken of the greater flexibility afforded.

Having reached this conclusion the type of girder to be used had to be decided on, and the double-double Bailey girders were selected because of their adaptability and high recovery value.

Description of the Construction

Seven double-double Bailey bridge girders each 60 feet long spanned across the walls in the long direction of the span, as indicated in Fig. 10, and were placed at 6 ft. 2 in. centres to suit the positions of the previously mentioned holes.

Underneath each girder was slung a pair of 7 in. $\times$ 3 in. R.S. channels, 7 in. back to back, attached to short pieces of 10 in. $\times$ $4\frac{1}{2}$-in. R.S. joist, which occupied the positions in the girders normally used by the bridge transoms. Standard Bailey transom clamps were used to retain the joists in position. The 7 in. $\times$ 3 in. channels now enabled 12—$1\frac{1}{2}$-in. dia. suspension rods supporting the remainder of the work to be placed at the desired positions along each girder.

From these rods were suspended the members supporting the 6-in. steel thermal shield plates as shown in Fig. 11. These were 9 in. $\times$ 9 in. tees, made by welding $\frac{1}{2}$ in.-thick plates together, and attaching them in the inverted position to gusset plates, which in turn were fixed to the suspension rods. The attachments were made by bolting through slotted holes, to permit movement due to expansion under the effect of temperature.

A system of temporary bracing was attached to the underside of the tees to ensure that the structure remained true and rigid during the erection of the 6-in. steel thermal shielding. The plates simply rested on the flanges of the tees without connection, sufficient clearance being provided to allow for expansion. The plates were lifted into their approximate positions by means of the permanent overhead crane which was in operation by this time. Their final locations were determined by holes in the graphite structure with which the holes in the plates had to register to within ± 0.03 inches and this fine adjustment was achieved by using jacking screws working through temporary cleats on the supporting tees. The 3-in. thick insulation boxes were then laid on top of the plating and connected together but not to the plating, care being taken that the holes registered with those in the thermal shield.

With the completion of the first stage the next main operation was the fixing of the permanent steel shutter plates and their supports. These supports were double $6 \times 4 \times \frac{1}{2}$-in. M.S. angles riveted together with a packing between them of the same thickness as the gusset on the suspension rods, the packings being kept short of the ends of the members so that the gusset plate could enter between the angles. The contact surfaces of the angles and packing were painted and brought together for riveting whilst the paint was still wet to provide a

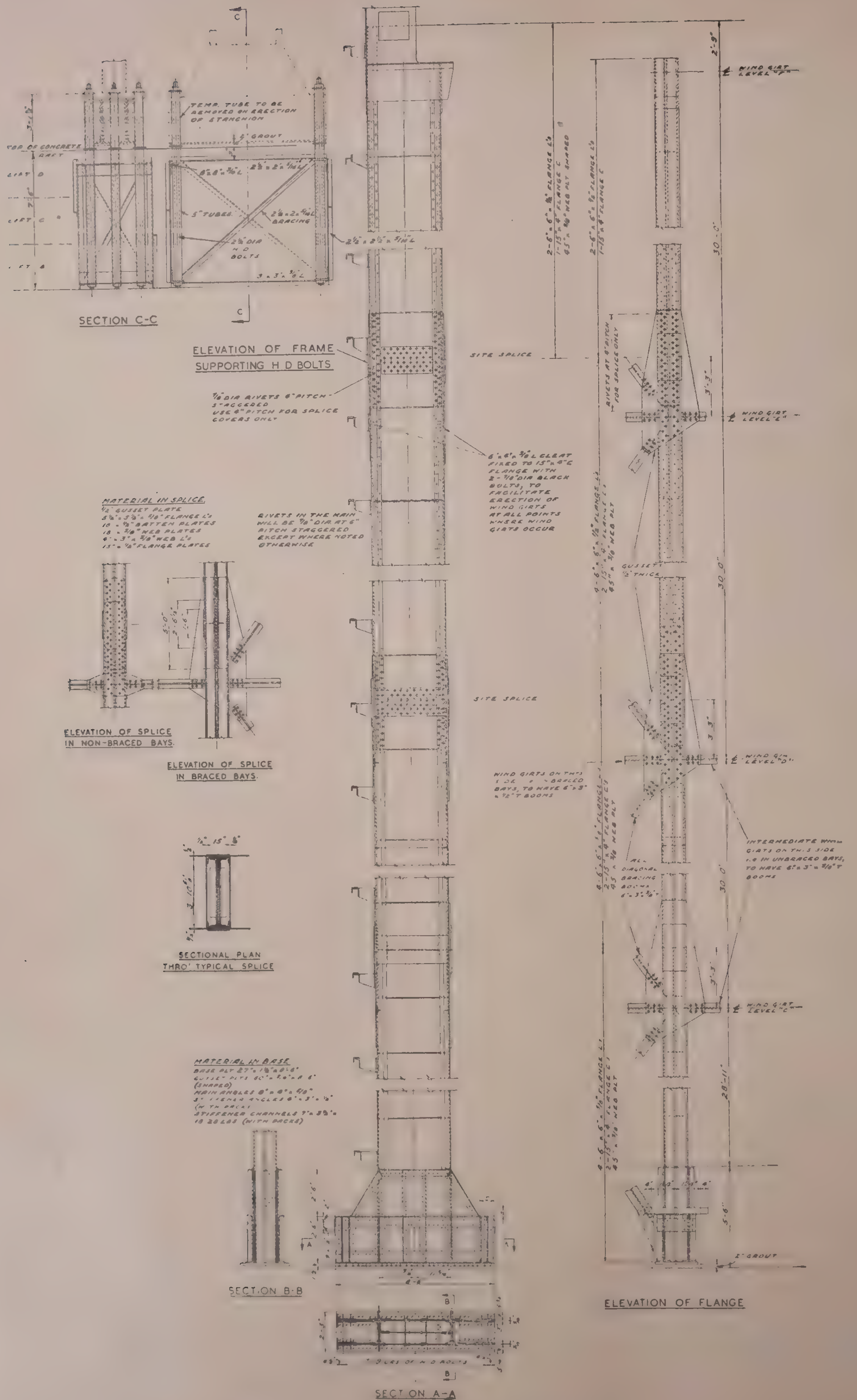

Fig. 8. Details of portal post up to frame level

grout tight seal. After riveting, 3-in. dia. holes at 9-in. centres were drilled through the combined thickness for the insertion of the lowest layer of reinforcement.

The permanent shuttering was next placed in position. This consisted of $\frac{3}{8}$ in. and $\frac{1}{2}$ in.-thick plates of size 6 ft. × 4 ft. and 6 ft. × 5 ft. 6 in., and again the holes had to be carefully aligned with those in the shielding and insulation. The plates were then clamped down preparatory to being welded together and to the supporting angles to provide a grout tight structure.

The welding was carried out by first welding the cover plates to the joints to provide long plates of the required width. This done, the welding of the plating to the supporting members commenced at the centre of each plate length and continued outwards to the ends. Intermittent welding of about 12-in. weld and 12-in. gap was used, the welders returning to complete the gaps.

The structure was now reasonably grout tight except for the points where the suspension rod connections

Fig. 9.—Erection of structural steelwork
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penetrated the plating, and it was decided here that a form of red lead jointing compound would suffice to make good at these points.

Adjusting the Level of the Structure so far Erected

There still remained the task of providing shuttering tubes to form the holes in the concrete through which the control of the many safety devices would operate, and since these holes had to be accurately formed so that they would register with similar holes in the graphite structure (which was not yet built), it was decided to check the structure for level so that an exact datum could be established.

It should be noted that at this stage the load being supported by the Bailey girders was about 350 lb. per sq. ft.

The hanger rod lengths were adjusted by screwing up the nuts until the required corrections were made to bring the underside of the thermal shield plates to the exact level above the base of the graphite. This level had to be maintained after the completion of the roof, no deflection below the horizontal being permissible, though a slight upward camber would not have been serious.

The formation of the many holes in the slab and the accurate positioning required for them in view of their intimate connection with the graphite structure called for special consideration (Fig. 12). These holes came into two separate categories ; the first category was dealt with by casting in steel tubes stepped in diameter to provide adequate shielding, into which were inserted smaller diameter tubes when the concrete work had been completed. The space between the sleeve and liner tubes then being filled with cement mortar. The close centres of the tubes in the second category prevented the use of sleeve tubes and it was therefore necessary to cast these tubes directly into the concrete, whilst still maintaining them very accurately in position. The tubes were therefore supported by light steel frames housed within the concrete thickness and welded to the shutter plates, adjustment being effected by means of set screws bearing on both the top and bottom ends of the tubes.

In order to ensure that the underside of the roof was truly level and did not deflect below the horizontal, it was now necessary to pre-camber the shuttering. The amount of the pre-camber was determined from test loading carried out at ground level on two double-double Bailey girders. These tests showed that the slip on the panel pins would all occur at loads less than those imposed by the thermal shield plating, and that the deflection of the girders thereafter would agree very closely with those calculated.

Once the tubes had been accurately set in position, the nuts at the upper end of the suspension rods were therefore tightened up so as to give a maximum pre-camber at the mid-point of the roof slab of $\frac{7}{8}$ in.

Concreting the Slab

The structure was now ready to receive the concrete forming the roof shield, which was to be poured in five lifts to a total thickness of 8 ft. 6 in. The first lift of $10\frac{1}{2}$ in., suitably reinforced, was carried by the permanent steel shuttering, and in its turn, became the permanent shutter supporting the second lift, which was 2 ft. 3 in. thick.

In pouring this and later lifts of concrete, provision was made for shrinkage and also for the deflection of the Bailey girders by leaving an annular gap between the supporting walls and the roof slab, which was not filled until at least three days after the main centre section of the roof slab had been cast.

A period of seven days was allowed to elapse between the pouring of the second lift and its being loaded up with the third lift which was 1 ft. 9 in. thick. The first two lifts then acted together to take a major share of the imposed load from the third lift by square spanning between the supporting walls, the balance of the load being carried by the Bailey girders.

Once the third lift had matured for seven days, the roof slab, now 4 ft. $10\frac{1}{2}$ in. thick, was capable of carrying all subsequent lifts of concrete, and the suspension rods were therefore cut, and much of the Bailey girder system removed. Provision was made against the rods pulling out under the weight of the thermal shield plating by welding to the rods small plates which were housed within the thickness of the third lift.

The deflections of the girders were checked after each day's concreting and after the cutting of the suspension rods, and it is interesting to record that at no point did the slab deflect below the horizontal, and that, at the critical portions of the structure where the graphite approached the roof, there remained a camber varying between 0.02 in. and 0.13 in.

The maximum recorded deflection of any girder was therefore $\frac{3}{4}$ in., excluding that due to slip on the panel

pins, and the maximum load carried by each girder was of the order of 170 tons.

Design of the Ventilation Shafts

Up to the level of the concrete roof over the Reactor, the base walls of the ventilation shaft constitute part of the concrete shield to the Reactor and are 5 feet thick. They therefore had to be treated on similar lines to the

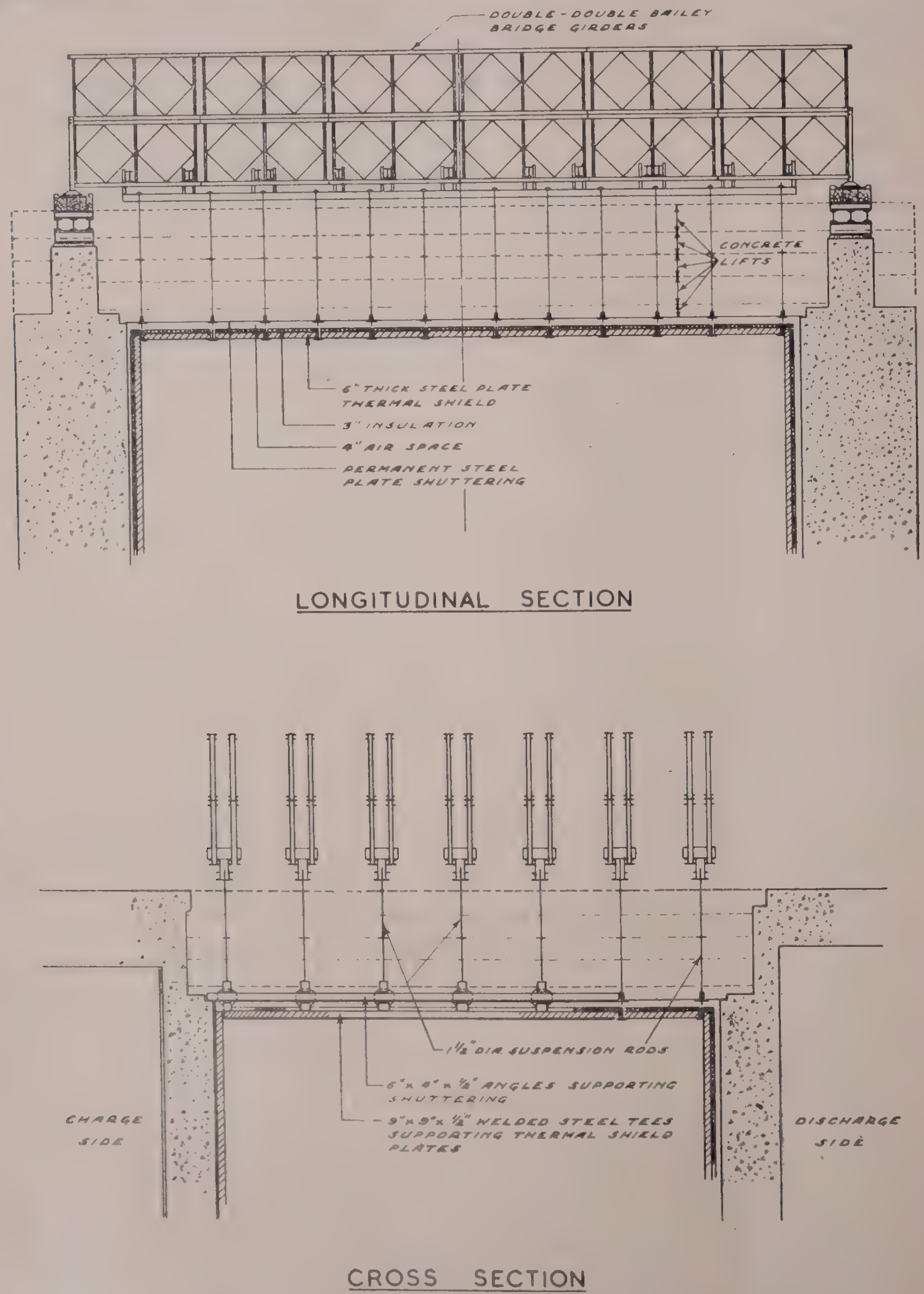

Fig. 10.—Sections through Bailey Bridge system

ress at the concrete [illegible] also did the plinth to the shaft which extended above this level to a height of 130 feet above ground level. From this level upwards the design of the shell was determined by structural considerations only, and it is with the work above this level that this section of the Paper is concerned.

Towards the end of 1948 the construction of the concrete shield walls was nearing completion, and arrangements were in hand for the removal of the scaffolding within the walls at the base of the shaft, and the substitution of a steel-framed internal tower from which construction of the shaft would be carried out. (Fig. 13.)

The shaft as then designed was 400 feet high above ground level, with provision for a further 50 feet if required, and it tapered from an inside diameter of 44 ft. at plinth level down to 35 ft. at the top. The shell was to be 15 in. thick at the plinth and 8 in. at the top, with concrete corbels formed at 30 ft. vertical intervals to support a refractory brick lining.

At this late stage information was received that rendered the provision of filters in the exhaust air system imperative, and a decision was taken after considerable investigation of alternative schemes, to locate these at the top of the shaft. This decision gave time for the development of the design of the filter gallery, while enabling the construction of the shaft to proceed with

minor modifications. The revised design had to take account of the fact that the contractors were committed to its construction from an internal steel tower, and any departure from this principle would have meant a serious set-back in the programme. The modifications introduced were therefore as follows :—

(a) The shaft was now to be parallel sided, the shell thickness tapering from 15 in. down to 12 in. at the underside of the diffuser section, where the shaft changes from a circular to a square cross-section.

(b) The refractory brick lining would be omitted to cut down the dust hazard, and metal insulation boxes substituted, filled with slag wool. These boxes would be supported on steel stanchions taking a bearing on the concrete corbels incorporated in the original design.

(c) The reinforcement in the shell was increased to take account of the new factors in the design.

In the meantime tests were carried out in a wind tunnel to determine an acceptable shape for the top of the shaft, which would give the required degree of dispersion into the atmosphere. From these tests it was determined that the filters must lie in a horizontal plane, and that the exhaust air must be discharged to atmosphere in a vertical direction.

It was desirable that the area occupied by the filters should be square in plan, and this made it necessary for the circular shape of the shaft to change to square at the filter gallery, and revert to circular in the top section.

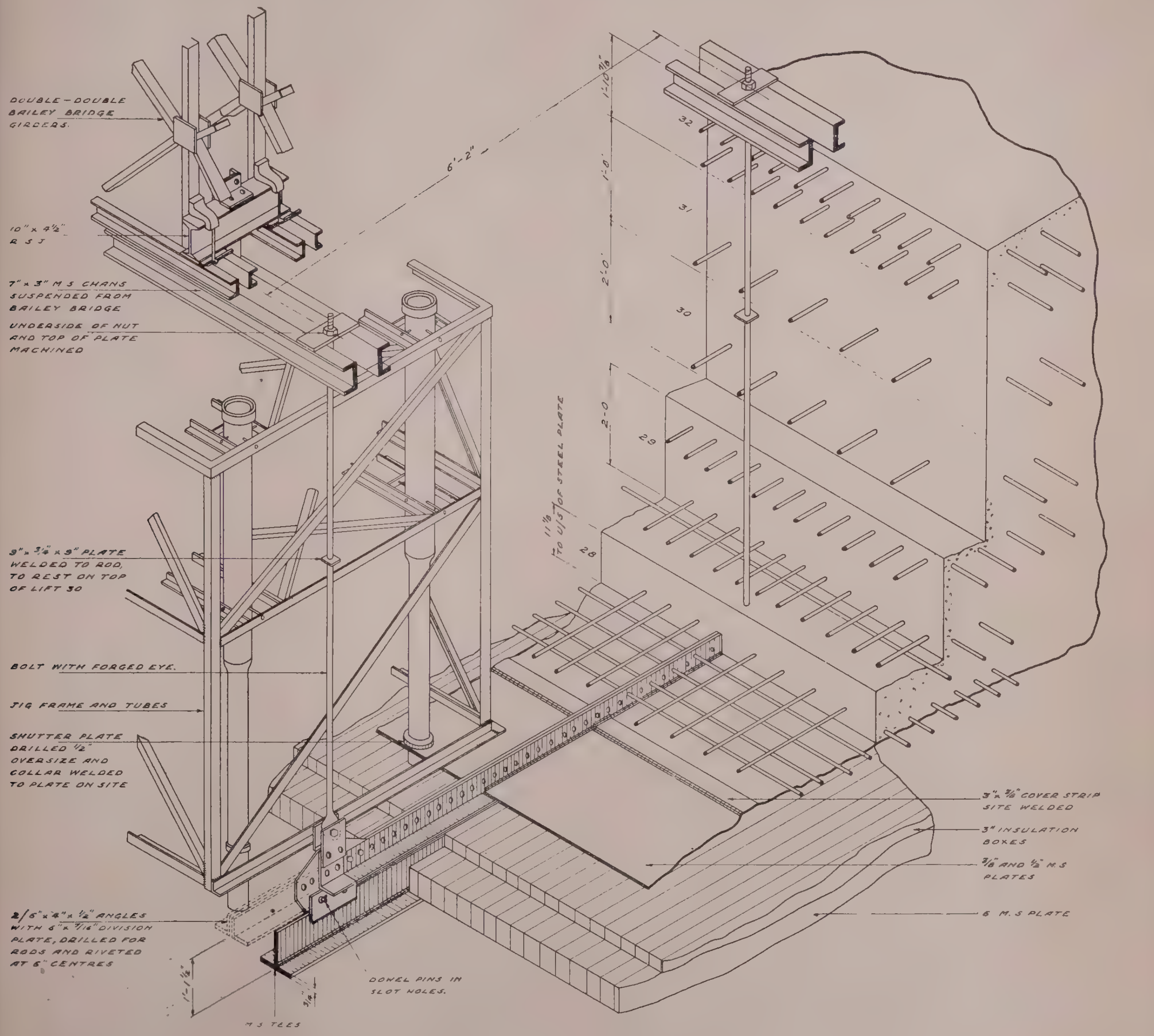

Fig. 11—Isometric diagram of Bailey Bridge system

Abrupt changes of section in the shaft were inadmissible from an air flow point of view, and the changes had therefore to take place gradually over appreciable heights. This involved the provision of sloping surfaces, whose inclination to the vertical varied from almost nothing at the mid-point of the sides of the square to a maximum at the corners.

It was felt that the following of such a shape with the concrete shell would lead to very intricate shuttering, and it was therefore decided to accept the severe temperature conditions arising at abrupt changes in the shell cross-section, by making all concrete surfaces either

vertical or horizontal. The sloping surfaces could then be formed in sheet metal supported off the steel framework to which the insulation boxes were to be fixed.

A filter gallery external to the shaft would be necessary for handling the filters, which would be slid into position along guides carried across the air stream on a system of steel lattice girders. The operation of discharging the filters would be carried out by allowing them to fall into a water trough in the gallery, whence they would be removed into lead containers for disposal.

Fig. 12.—Some holes through concrete roof over reactor
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Fig. 13.—Cylindrical section of ventilation shaft during construction
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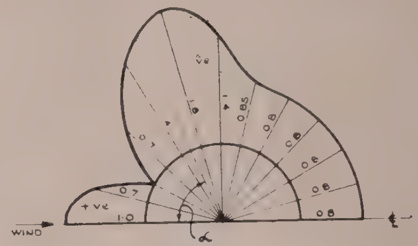
The gallery would be pressurised to prevent the leakage of contaminated air from the shaft into the gallery, and an air lock would therefore be provided where the external lift shaft entered it. This lift, which was to convey the filters in their containers down to ground level, was housed in a steel-framed shaft, clad in asbestos cement sheeting, and stayed back to the main concrete shell. This lift shaft was also used to house the services and air ducts which had to be conveyed to gallery level.

Cylindrical Shaft

The design of the cylindrical shaft was conditioned by temperature effects, vertical loads, and overturning and secondary bending moments caused by the wind forces. The design of the shell for temperature effects did not present any unusual problems, but the wind forces and their effects were more difficult to assess owing to the



WIND PRESSURE DISTRIBUTION

Bending Moments in Horizontal Ring (Symmetrical about ℄).								
α	0°	15°	45°	75°	105°	135°	165°	180°
M	$-0{\cdot}257pR^2$	$-0{\cdot}233pR^2$	$+0{\cdot}083pR^2$	$+0{\cdot}292pR^2$	$+0{\cdot}157pR^2$	$-0{\cdot}068pR^2$	$-0{\cdot}217pR^2$	$-0{\cdot}257pR^2$

Positive values indicate tension on outside of shell.
Negative values indicate tension on inside of shell.

Fig. 14.—Wind pressure distribution on cylindrical shaft

lack of research on tall chimneys. It was therefore decided to carry out the design on the following basis :—

(a) Wind forces causing overturning would be based on Chapter V of the Code of Practice, a mean velocity of 85 m.p.h. being assumed.

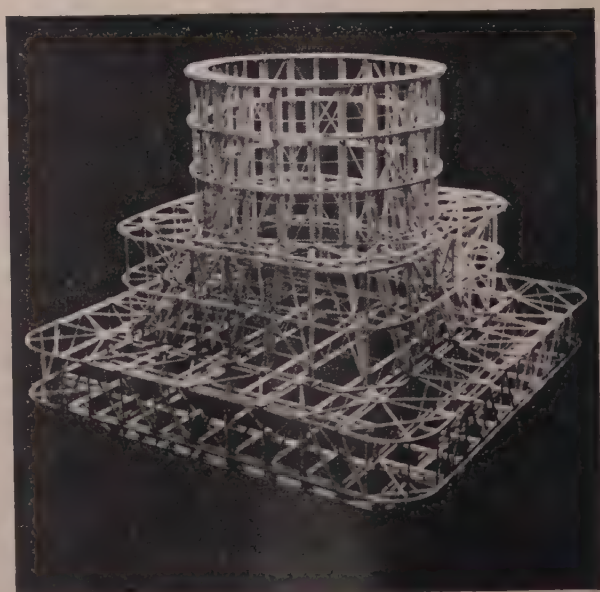


Fig. 15.—Model of framework at top of shaft
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(b) Wind pressures at any level would be derived from Stanton's Formula.

(c) The pressure distribution would be based on the work of Dryden and Hill, as reported in their Paper, "Wind Pressures on Circular Cylinders and Chimneys." (Research Paper No. 221, 1930. U.S. Dept. of Commerce. Bureau of Standards.) This distribution, and the moments derived from it, are shown in Fig. 14.

The actual wind pressures used in the design when considering overturning, were taken as 45 lb. per sq. ft. with a form factor of 0.75. While the peak pressure at the top of the ventilation shaft was taken as 55 lb. per sq. ft. in assessing the secondary bending effects.

The stress in the reinforcement arising from all effects was limited to 12,000 lb. per sq. in., and the average

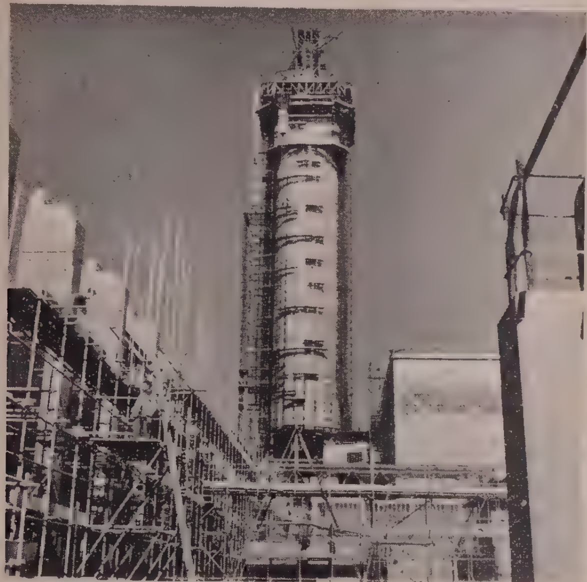

Fig. 16.—Framework being erected at top of shaft

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quantity of reinforcement required was therefore as follows :—

Vertically in both faces.	$\frac{5}{8}$ in. dia. at 6 in. crs.
Horizontally, inner face.	$\frac{5}{8}$ in. dia. at 6 in. crs.
Horizontally, outer face.	$\frac{7}{8}$ in. dia. at 6 in. crs.

Laps in the vertical bars were located in the normal way to follow a series of spirals, so that not more than one quarter of the laps occurred in any one lift of concrete.

Square Diffuser Section

That part of the shaft between the cylindrical section and the filter gallery formed the height over which the transition from a circular to a square cross-section was effected, and is described as the diffuser section. It is illustrated in the model shown in Fig. 15 and also in the photograph of the shaft under construction, Fig. 16.

Basically the design of the diffuser section consisted in the formation of a reinforced concrete collar at the top of the cylindrical shaft, which would then support the 12 in.-thick reinforced concrete walls to this section. (Fig. 17.) To overcome the temperature effects in this collar it was designed as a honeycomb, cooling being provided by natural convection, for which air entered the honeycomb through holes left in the underside and passed upwards along the back of the insulation boxes. The design allowed for the temporary support of this collar while the concrete was developing in strength, these temporary supports being provided in structural steelwork resting on the cylindrical shell and tailed back to the central steel tower. As these temporary supports were capable of supporting not merely the collar but also the next two lifts of concrete in the walls, the time of construction of the latter was available

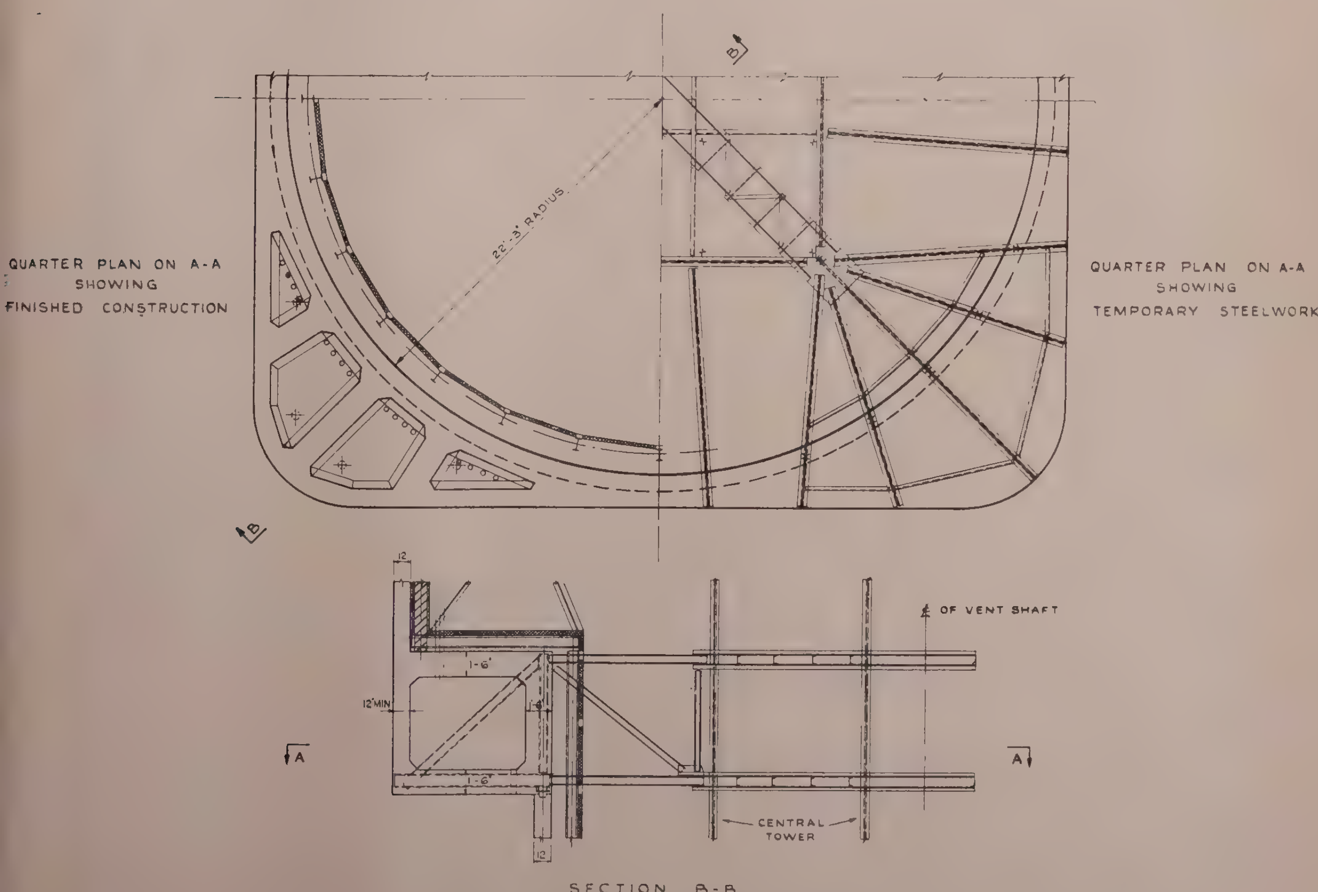

Fig. 17.—Details of diffuser section

as curing time and prevented any hiatus in the construction programme. Once these two lifts were cast the temporary supports were burnt off at the face of the concrete shell.

Filter Gallery and Concentrator Sections

The application of a similar design for cantilevering the filter gallery off the shell and tailing it down to the central tower was considered, but the functional requirements of the gallery rendered this impracticable. A steel framework was therefore designed, taking a bearing on the walls of the diffuser section.

By setting the bottom boom of the main lattice girders one foot above the top of these walls, a temporary platform could be introduced extending to the full limit of the gallery. This platform consisted of steel joists fixed back to the central tower, and supported a timber deck bounded by a safety barrier at its extreme edge. Being 75 feet square it not only provided a considerable degree of safety but also did much to destroy the impression of height for those working at this level.

The main lattice girders were ultimately housed within the reinforced concrete walls between the gallery and the air shaft, the weight of concrete in the cantilever sections being reduced by fixing expanded metal frames to these sections to form an inner shutter and thereby restrict the casing thickness to 6 inches. The girders lying within the outer walls to the gallery supported $4\frac{1}{2}$ in. of reinforced engineering brickwork externally and an inner lining of sheet steel designed to resist and contain the air pressure within the gallery.

Thermal insulation took the form of insulation boxes [illegible] design to the rest of the shaft, which were [illegible] steel members tied back to the main steel [illegible] being made for differential thermal [illegible] these members and the main frame.

[illegible] of the filter gallery were of normal [illegible] rete construction, the former [illegible] which the bogies carrying the [illegible]. Charging machines, for [illegible] hannels spanning the air [illegible], also operated in the gallery, the filters being [illegible] ed through slots in the shaft walls. Two complete [illegible] were therefore permanently shuttered in steel plate fixed to the main steel frame, between which were welded steel linings for these sl[illegible].

The concentrator section above filter gallery level, in which the squar[illegible] verts to a circular one, was also framed in steel, clad externally in reinforced engineering brickwork, and lined with insulation boxes. It terminates in a mild steel capping plate below which the cooling air from behind the insulation boxes is turned back into the main air stream.

Construction of the Ventilation Shafts

As shown in Fig. 13 the cylindrical section of the shaft was contructed from a central steel tower in concrete lifts of 3 feet, the shutters being lifted by blocks suspended from jibs fixed to the tower. Within the steel tower were housed two hoists and two lifting wells, for the transport of concrete in barrows, reinforcement, the steel framing for insulation boxes, and the boxes themselves.

When the shaft was re-designed to incorporate the filter gallery it was realised that the transport of all material to this level could not be achieved in the time by these two hoists alone. In all, 1,900 tons of material were required between the underside of the diffuser section and the top of the shaft. In addition, it was felt that some form of external scaffold would be needed for access to shuttering on the soffits of all offsets, even though there would be no requirement for it to carry the weight of any concrete placed.

An external scaffold, to which were attached five hoists, was therefore erected, and caught up with the level at which concrete was being placed just as the cylindrical section was completed. These additional hoists were primarily intended to deliver material to the temporary working platform at the underside of the steel framing to the gallery. Only one of them was carried up to gallery roof level, owing to the restriction which such hoists caused in the circulation within the gallery itself.

In the meantime the steel framing for the filter gallery and concentrator sections was being fabricated, and was pre-erected at the fabricator's works (Fig. 18) by the gang that was later to erect it at the top of the shaft. The erection of this frame at the top of the shaft was carried out by means of four jib cranes fixed to the central tower, as can be seen in Fig. 16, the larger items being assembled and riveted together on the temporary working platform.

At this stage the 330 feet of steel tower up to the underside of the diffuser section was dismantled for re-use on the second ventilation shaft, the top lengths being supported off the concrete collar at this level and

Fig. 18.—Steel framework pre-assembled at works

remaining in position until the lining of the shaft was complete.

While this work was going on, the lift shaft framing was being erected. As none of the facilities available for the construction of the ventilation shaft could be spared for this operation, the erection had to be carried out as if the lift shaft were an independent pylon, except for the lateral support afforded by the ventilation shaft. The external scaffold was adapted to enclose the lift shaft and was then used by both painters and sheeters.

Conclusion

Work had started on the excavation for the foundations to the first Reactor in October, 1947, and by July, 1950, the work was complete and the Reactor could be put into operation. The second Reactor was started some six months behind the first, and though used as a reservoir of labour, kept in step with the first one so that it was completed in December, 1950.

Throughout the construction of the first Reactor the design was being continuously developed, and it was with difficulty that detail drawings of the work were prepared in sufficient time to prevent the work on site being held up. At no time were drawings of the concrete work issued to the contractors much more than six weeks prior to construction, and in many cases the interval was much less. This gave little opportunity to the Resident Engineer and the contractors for detailed planning, planning which involved a considerable degree of dove-tailing of different trades and operations in order to achieve the speed of construction required. The fact that they made very effective use of such limited opportunities enabled the project to be completed within a week or so of the original target date.

Much was learnt from the construction of these Reactors which has led to improvements in the design and also in the construction of subsequent Reactors. These improvements are mostly in detail points, but are leading to substantial reductions in capital cost, particularly where they are associated with the achievement of the high standards of accuracy demanded in this type of work.

The author looks forward to the widespread development of Nuclear Reactors for power production, when, it is expected, the technology associated with this work will be rapidly developed. Great advances have already been made, but it is to be remembered that any mistakes leading to accidents with these potentially dangerous sources of power must inevitably deal a mortal blow to their development, and it is therefore very important to be successful and not to attempt too much at each step forward.

Acknowledgements

The author is indebted to Sir Charles Mole, Director-General of Works, Ministry of Works, and to Sir Christopher Hinton, Managing-Director of the Industrial Group of the Department of Atomic Energy, for permission to present this Paper.

His thanks are also due to his colleagues for their help in its preparation, in particular to Mr. P. A. Badland, A.M.I.Struct.E., A.M.I.C.E., Mr. S. G. Silhan, A.M.I.Struct.E., Mr. G. E. Crowe, A.M.I.Struct.E., and Mr. W. H. Chilton.

Book Reviews

Structural Theory and Design, by J. McHardy Young. (London : Crosby Lockwood.) One-Volume Ed. 1954, 599 plus ix pp., $9\frac{3}{4}$ in. × $6\frac{1}{4}$ in. 30s.

A one-volume edition of this book, first published in 1950, now appears for the convenience of readers, at a reduced cost. The book, which combines the fundamental theory of structures with the practical knowledge of structural design, has become a popular textbook since its first appearance.

Design of Structural Welded Details, by F. H. Abrahams. (Richmond, Surrey : The Association of Engineering and Shipbuilding Draughtsmen, 1953.) 76 pp., $8\frac{1}{4}$ in. × $5\frac{1}{2}$ in. 3s.

This booklet is divided into two parts, the first giving general notes on welding and the second dealing with design of details. It is assumed that the reader possesses a knowledge of elementary theory of structures, but most of the calculations are of a simple nature. The text is well illustrated by forty-seven figures and contains useful tables. A short bibliography is included.

Solution of Problems in Strength of Materials, by S. A. Urry, B.Sc.(Eng.). (London : Pitman, 1953.) 381 pp. $7\frac{1}{2}$ in. × 5 in. 20s.

Students of a former generation will recall the late G. W. Bird's useful book of worked and unworked examples in Strength of Materials. Urry's book, admirably produced and reasonably priced, contains 140 examples fully worked in lucid style, and 194 unworked examples, with answers, covering the usual topics in simple and complex stress and strain, bending, torsion, thin and thick shells, springs, and strain energy. In addition, there are two appendices, on anlysis of experimental results, and moments of area, and an index. Very commendable features include the insistence on correct units, an explicit discussion of Mohr's theorems, and a comprehensive treatment of Mohr's stress circles. The standard is that of Higher National Certificate, but Degree students, especially first year, have also been catered for.

Bird gave a brief recapitulation of theoretical results, whereas Urry has incorporated, in the form of problem and solution, all the theory required, so that those whose primary concern is the passing of an examination may be tempted to buy and use the book as a convenient substitute for, instead of an excellent and necessary supplement to, the study of the inspiring works of writers such as Salmon who, like Ewing, combined masterly erudition with faultless clarity of style. Such substitution is, undoubtedly, not intended, but the possibility, with consequent detriment to the best interests of students, cannot be excluded.

F. A. G.

Welding Technology, by F. Koenigsberger. (London: Cleaver-Hume Press, Ltd., Second Edition, 1953 ; pp. 341 + viii, $8\frac{1}{2}$ in. × $5\frac{1}{2}$ in. 25s.)

The second edition of this well-known manual on the many aspects of welding technology follows the same general layout as the first edition, but the whole work has been brought up to date, and completed by the addition of information on recently developed methods, and references to recent literature.

The book is a remarkable and useful compendium of contemporary information on the techniques of welding in all its aspects, apart from design.

Examples of Precast Ferro-Concrete Construction in France*

By N. Esquillan, M.Soc.C.E.(France)

During the course of the past five years, precast ferro-concrete construction in France has made considerable progress, and, for various reasons now appears to be in general use. It possesses the following advantages :—

(1) A saving in shuttering by the use of a reduced set of moulds, solid and accurate.

(2) A simplified form of shuttering ; for instance there is a considerable difference between a portal built *in situ*, and the same one cast flat on the ground or, preferably on a concrete floor where only lateral shuttering is necessary, and recoverable a short time later.

(3) No need to search for highly skilled carpenters, often so difficult to find nowadays.

(4) The actual pouring of the concrete need not take place on scaffoldings ; strains, especially at great heights, are detrimental to the accuracy of slender frameworks.

(5) The various concrete members can be cast in the most suitable positions so as to ensure best quality concrete. For instance, thin vertical slabs can be cast flat on the ground.

(6) Special treatments (steam, vacuum, vibrations) can be more easily applied.

(7) The concrete members can be readily tested at ground level and the factor of safety of a series of similar elements ascertained with accuracy.

(8) It is possible to obtain dense and homogeneous concrete members of accurate dimensions and in perfect uniformity.

(9) The weight of the concrete members can be reduced, either because of the increased quality of the concrete which warrants higher stresses, or as a result of tests which can limit the factor of safety to a relatively low value.

(10) Supervision at ground level is easier to carry out, and the chances of accidents very much reduced.

(11) Weather conditions do not affect the work since it can be carried out under cover. During frosty days, the concrete can be poured in warm and closed sheds, whilst in hot weather the work can proceed under cool and shaded conditions.

(12) The time of construction can be reduced. For instance, the various members of the superstructure can be prepared in anticipation, during excavation and foundation work.

[illegible] The work can proceed during daily stoppages caused, for instance, by tides or traffic interruptions, and [illegible]n during seasonal stoppages caused, for instance, by [illegible]ds or continuous frosty periods in cold climates.

[illegible]des of labour can be utilised in [illegible] appropriate manner.

[illegible] work can be prepared and [illegible] increasing efficiency.

[illegible] us to cross over from the handicraft stage to the mass production of the various components involved.

Yet, it has its drawbacks ; it demands more elaborate designs, and it can only be worthwhile in cases when a great number of identical members are involved.

Problems of stockpiling and of horizontal and vertical handling have to be solved. This calls for the special site equipment which has been for so long the prerogative of structural steelwork.

All this is expensive, and the risk of damaging the precast concrete members is always present, from a simple crack to the total loss of a member, often an important one.

Finally, and especially, in the case of ferro-concrete, the correct assembly of the various components is not always easy to carry out economically.

We shall now endeavour to illustrate these facts by bringing to notice five varied examples of application. We have chosen four of them in Boulogne (Fig. 1) as these may be familiar to many British engineers. They are :—

(I) The marine station cutwater.
(II) The marine station landing stage.
(III) The Lampe bridge over the River Liane.
(IV) The buildings on the Gambetta harbour front.
(V) The fifth and last example deals with the hangar at the Marignane Airport near Marseilles, which has already been described during the Cambridge Congress in August, 1952.

I. The Boulogne Marine Station Cutwater

The marine station cutwater was built in 1928, destroyed in 1944 and rebuilt in 1950.

The tidal range is 30 feet, and, even during neap tides, an important part of the work was under water twice a day. To abate such discontinuities, so detrimental to the smooth running of the work, to avoid wire-brushing the reinforcement and cleaning the shuttering each time before pouring the concrete, and to ensure strong, and especially, dense concrete, it was decided to resort to prefabrication.

In order to spread the cost of the special equipment required over as large a number of units as possible, prefabrication was extended to the parts always above water. The deck consisted of precast concrete ribbed slabs.

The result was a reduction in time of construction and, consequently, a saving in overhead expenses.

All concrete work liable to remain under water was carried out with a special slow setting rapid hardening cement which offered high resistance to the chemical action of the sea water in the harbour.

We had to solve the problem of joining the various precast concrete members together.

Some of the longitudinal bars reached a diameter of one inch in three lower beams. It was necessary to provide supports for anchorage and connecting operations, and to ensure correct overlapping of the reinforcement in order to withstand the stresses resulting from negative moments. Straight bars would have been too short to provide the necessary overlap. The classical type of hooked ends had to be avoided as it would interfere with the placing of the precast members, and

Paper to be read before a Joint Meeting of the Institution of Structural [illegible] of the Societe des Ingenieurs Civils de France at 11, Upper Belgrave Street, London, S.W.1, on Thursday, November 11th, 1954, at 6 p.m.

often gets caught in the slings during hoisting, or in the reinforcement of the adjacent members.

It was decided to loop the ends and bend part of the reinforcement so that, when the second member was brought down contact was made at one point, and not along the whole length of the bar. This arrangement, supplemented by a staggering in plan of the reinforcement, was very helpful when placing the precast members in position (Fig. 2).

The operation was carried out by means of a 1600 ft./ton floating derrick and a 300 ft./ton transporter crane.

The "T" shaped main beams are 22 ft. apart. Most of them are bressummers continuous over five spans of 40 ft., 35 ft., 50 ft., 52 ft., and 51 ft.

The secondary beams are rectangular in section 12 × 33 inches. They were prefabricated and laid on the main beams. They support the 6¼ inch-thick deck slabs to which they are joined. (Figs. 3 and 4.)

In short, the designs are a translation into ferro-concrete of the usual timber structures. We have made use of the architectural treatment of this constructional arrangement.

Fig. 1.—General view of Boulogne Harbour showing the Marine Station cutwater and landing stage, with the Lampe bridge and the Gambetta Harbour front buildings

The weight of the prefabricated elements varies from 2 to 27 tons.

II. The Boulogne Marine Landing Stage

The reconstruction of the marine station involved the following civil engineering work :—

(1) A ferro-concrete landing stage.
(2) A railway track.
(3) A roadway with a double track gradient.
(4) A track for high gantry type of jib cranes.
(5) Shelter awnings.
(6) Station buildings with restaurant.
(7) Customs buildings.

The ferro-concrete landing stage is 600 feet long and 370 feet wide. It was designed to be completed in record time. To obtain this result it appeared necessary to apply methods of prefabrication which would not be influenced by seasonal variations, such as those which prevailed during the winter of 1950/51, and remain independent of the progress of foundation work.

We therefore prefabricated systematically all beams and deck slabs where identical members could be used.

The deck is designed to carry everywhere the usual bridge load, namely, a uniform load of 245 lb./sq. ft., and it includes two zones, one where only 15 ton maximum vehicles are admitted, and the other where 25 ton trucks are allowed.

The main beams are designed to carry these loads, and consequently, the precast concrete members forming the deck and the secondary beams can be laid as soon as the main beams are ready.

The deck and secondary beams are interdependent and a few bars tie the latter to the main beams. The continuity of the secondary beams is ensured at the supports by means of capping bars, hoisted with the beam and made to slide into the reinforcement network of the next beam. The junction is then concreted with the deck slabs.

The continuity of the deck slabs is ensured by means of a reduced overlapping of the reinforcement, followed by lap welding, conveniently carried out from above and ensuring satisfactory results.

A series of tension tests was carried out on medium hard steel bars which were not embedded in concrete.

The overlapping was 17 diameters and the ends were lap welded. The breaking stress varied between 78,000 and 93,000 lb./sq. in. The break occurred in the bars, and not at the welds which were designed to withstand higher stresses.

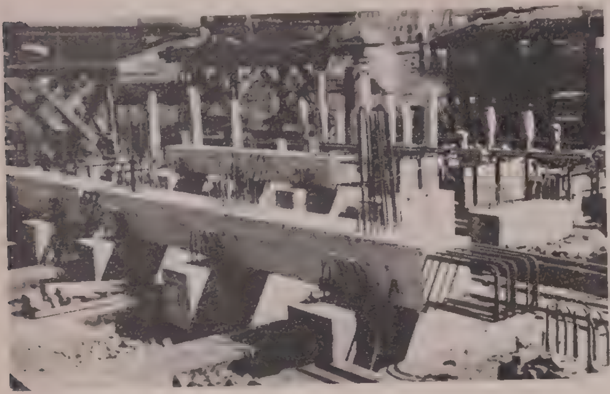

Fig. 2.—Boulogne Marine Station. Junction of precast concrete beams

Such tests were unfavourable, since the lever arm resulting from the eccentricity of the tensile forces adds secondary bending stresses. They were followed by similar tests, but, this time, the bars were embedded in concrete in the same positions they would occupy in the final design. The concrete cracked when the stress on the steel reached 62,000 lb./sq. in. and the break in the steel occurred, as in the previous tests, for stresses above 78,000 lb./sq. in.

Fig. 3.—Boulogne Marine Station landing stage under construction. Prefabricated deck slabs and beams

The main features of execution and positioning the precast beams and slabs were as follows :—

Each slab measured 8 × 21 feet and the only shuttering necessary consisted of steel lateral moulds provided [illegible] to maintain constant alignment of the [illegible]

[illegible] cast on a flat and hard surface. [illegible] dening, shuttering oil was sprayed [illegible] a new set of slabs cast above [illegible] of each slab, four 16-inch- [illegible] blown up by compressed [illegible] n required, or, if need be, injected with water under pressure. They served as jacks and were held in position by means of vertical [illegible] centre of the bags and [illegible]

The precast secondary beams were mass-produced on a flat working surface with timber side forms. The intervals between the beams were such that after the concrete had set the forms could be moved enabling a new set of beams to be cast in these intervals.

The handling equipment included a 90-ton jib crane designed to lift 6 tons at 50 feet radius, a smaller crane lifting one ton at 50 feet and another crane lifting $1\frac{1}{2}$ tons at 43 feet. Work handled from the sea was hoisted by means of a Turney floating derrick designed to lift 20 tons at 50 feet.

The 475 deck slabs required were encased in special protecting frames during handling and hoisting. They were laid in position at the maximum rate of 50 slabs per day.

During the period from March 9th to June 26th, 1951, the total area covered reached 113,000 sq. ft., including 90,000 sq. ft. of the deck and railway track.

III. The Lampe Bridge over the River Liane

This new ferro-concrete bridge is 325 feet long with five 65-foot independent spans. The total width is

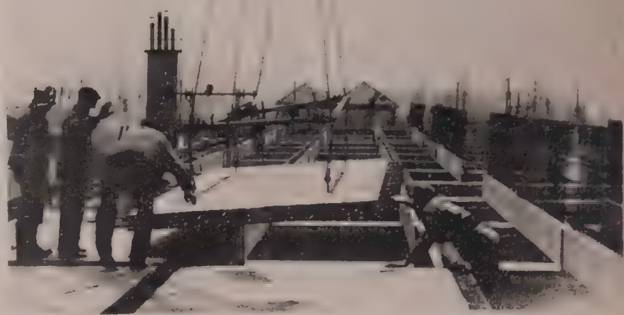

Fig. 4.—Boulogne Marine Station landing stage under construction. Laying deck slabs into position

76 feet and includes a 40-foot roadway with two islands, two 6-foot cycle tracks and two footpaths 9 feet wide.

Each span consists of four straight girders braced by three cross-beams which are extended on each side of the bridge as cantilevers to support the two outer beams under the guard-rail.

The piers consist of two trapezodial shaped walls connected by a spandril wall with upper and lower

Fig. 5.—Lampe bridge. Hoisting of capping units to caisson

struts. Each pier is founded on two circular caissons sunk by open grab methods.

The parts of the work which were prefabricated reached 27 per cent. for the piers and 80 per cent. for the roadway. All the precast members were made on

the banks of the River Liane, either with steel moulds for the beams, or concrete moulds in the case of the two outer beams ; a 50-ton floating derrick was used for handling and hoisting operations.

The bridge was built in the following 11 operations :—

1. Placing the caissons with the floating derrick.
2. Driving the caisson by open grab methods and filling lower part with concrete.
3. Placing of shuttering and reinforcement for lower struts, and capping units to caisson (Fig. 5).
4. Filling with concrete top part of caisson and concreting lower strut. Placing the reinforcement for the spandril wall and upper strut.
5. Concreting the spandril walls and upper strut.
6. Straight girders placed in position (Figs. 6 and 7).
7. Placing the cross-beams in position with their cantilevers partly concreted, leaving stub bars for future connection with the outer beams.

Fig. 6.—Lampe bridge. Hoisting of straight girders

8. Laying the precast concrete slabs of the roadway (Fig. 8).
9. Joining the slabs together, and with the girders.
10. Placing the outer beams in position and completing the cantilevers.
11. Laying the precast concrete slabs for the gutters and the footpath. Finishing work.

Fig. 7.—Lampe bridge. Placing straight girder into position

IV. The Buildings on the Gambetta Harbour Front

Four buildings are in course of construction on the quai Gambetta in Boulogne. Each one is 131 feet long (Fig. 9), 46 feet wide and 131 feet tall. Each floor space covers an area of 6000 sq. ft.

They consist of a basement, a ground floor to be used as business premises, and 11 floors including nine five-room flats.

The ferro-concrete skeleton entails precast concrete lattice work embedded in the partition walls up to the

Fig. 8.—Lampe bridge. Laying deck slabs into position

seventh floor. Above this floor, the columns and beams are designed as portals.

The columns were cast *in situ* along traditional lines and the remainder of the work was almost entirely constructed of precast concrete members.

Each building had its own jib crane for hoisting operations, and no general scaffolding was used, either

Fig. 9.—Buildings on the Gambetta Harbour front

outside or inside the buildings ; only a few stays here and there, and some access gangways. The work proceeded in the following stages :

(1) Preparation of the reinforcement and shuttering of the columns, followed by concreting. The columns were standardized in size so as to use the same shuttering over and over again.

(2) Placing of the precast concrete beams. The columns were set out in a 12×15 ft. pattern, and then followed the placing of the precast concrete main beams (Fig. 10) and bressummers which were U-shaped to form shuttering.

(3) Placing of the precast concrete floor members. The first floors were built by laying the 11-foot-long precast concrete beams one by one followed by the hollow tiles acting as shuttering and ensuring heat and sound insulation at the same time. The $1\frac{1}{2}$-inch-thick floor slab was then concreted in one operation.

Later on, we made precast members consisting of three or four beams with the hollow tiles and the $1\frac{1}{2}$-inch

concrete slabs attached. The junction of these large precast elements (Fig. 11) were the only parts to be cast *in situ*. We then made better use of the cranes and the men could rely at once on a solid working platform.

In winter, these elements were steam-heated to accelerate the re-use of the moulds.

(4) Precast members of the frontage.

They were specially treated, for decoration purposes, with coloured aggregate and bush hammering, etc. They were prepared flat on the floors and in line with their final position. The moulds were very gently sloped, however, to facilitate the washing of the face which would later be exposed to view.

After sufficient hardening of the concrete, the slabs were tilted and joined in position with the adjacent members. The breast-walls were similarly prepared. (Fig. 12.)

After two buildings had been erected, the speed of construction increased. A whole floor with its columns could be carried out in 12 days, and since the work proceeded simultaneously on two buildings at a time, this meant a floor area of 10,000 sq. ft. a day.

As in the case of industrial work, prefabrication coupled with rational organisation can be fully effective only when a great number of identical products are involved.

V. The Hangar at Marignane Airport

(1) Design of the Roof

In the short time available it is only possible to give you a general idea of the roof which presents original features in view of its large span, its design and method of construction. The two salient features are :—

(1) A record in the span of subtended thin arches, since the distance between supports is 333 ft.

(2) The construction of the roof which was carried out entirely at ground level and subsequently raised to a height of 62 ft., thus creating a new advance in the prefabrication of large structures and their erection by mechanical means.

The problem consisted in covering a free area of 197×328 ft. in each of the two bays of the hangar. The design was directed towards a complete utilisation of materials to find a self-supporting roof. Ferro-concrete arches were considered ideal as a "self-supporting" system to bridge great spans, but they had to be shaped suitably to act, at the same time, as a "roofing system."

The adoption of thin double curvature shells supplied a rational and economical s[illegible] the problem. They combine rigidity with lightness, and their shape reduces the chances of buckling and cracking. (Fig. 13.)

The basic element of the roof is therefore, a long "wave" of 333-ft. span. The neutral axis is the funicular polygon of the permanent loads, and resembles closely the arc of a 375-ft. radius circle with a rise of 40 ft.

For 262 ft. of its length, the cross-section of each wave is 32 ft. wide and consists of a thin shell, less than two and a half inches thick, and shaped in the form of an arc of a circle with a seven-foot rise.

Fig. 11.—Placing of the precast concrete floor members

Fig. 10.—Placing of the precast concrete beams

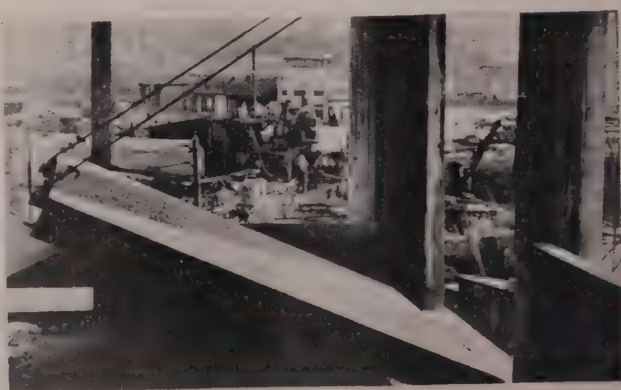

Fig. 12.—Tilting of frontage slabs

The junction of two successive waves forms the valley-channel of the roof, strengthened by an ogee-moulding to give each wave the necessary inertia and to place the centre of gravity in a favourable position.

Spandrils placed every 33 ft. along the length of the wave help to maintain the rigidity of the cross-section.

Towards the end of the wave, the thin shell is designed to play the part of the classical type of springings. The

difficulty, however, was to concentrate the loads on the centre lines of the columns and tie-beams and to avoid important bending moments in any part of the wave. over the whole area. It may be of interest to note that this same wave could span 540 feet if it were produced to ground level and anchored into suitable abutments.

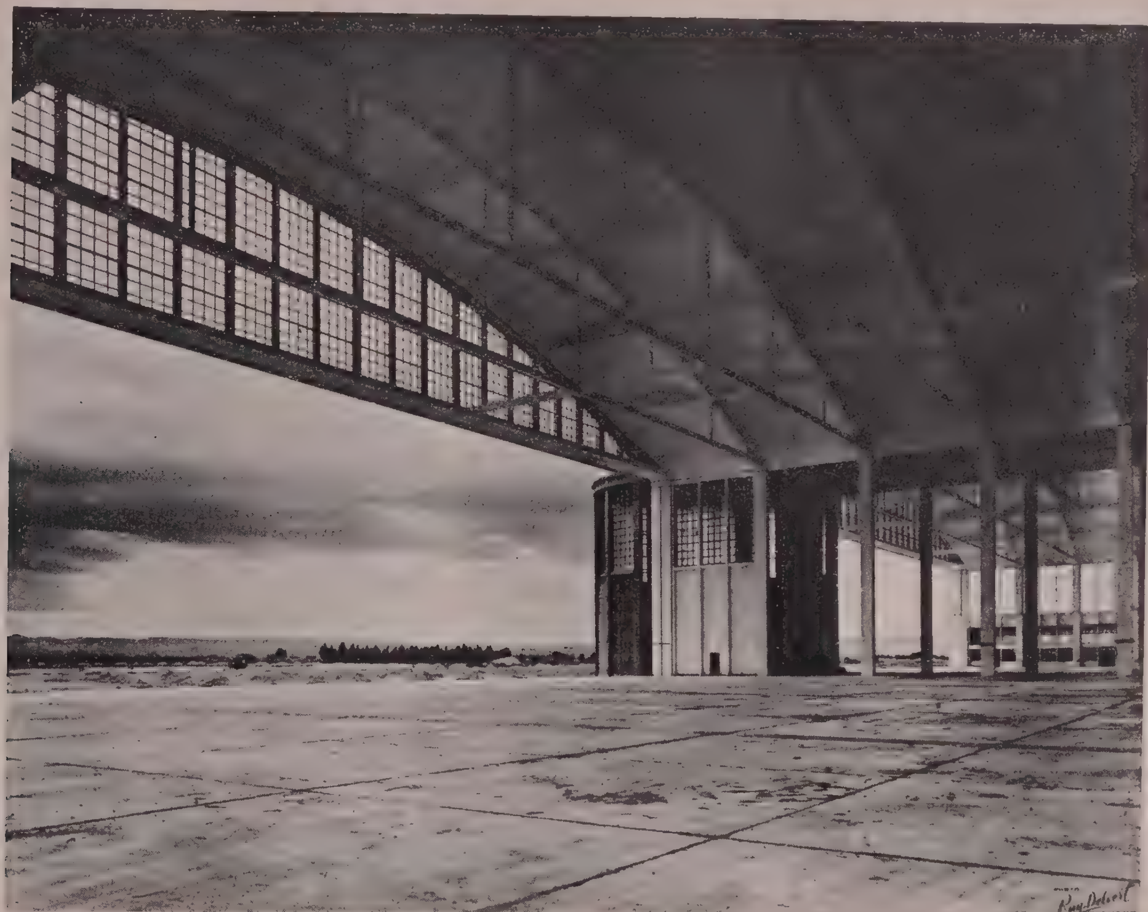

Fig. 13.—The hangar at Marignane Airport. Interior view.

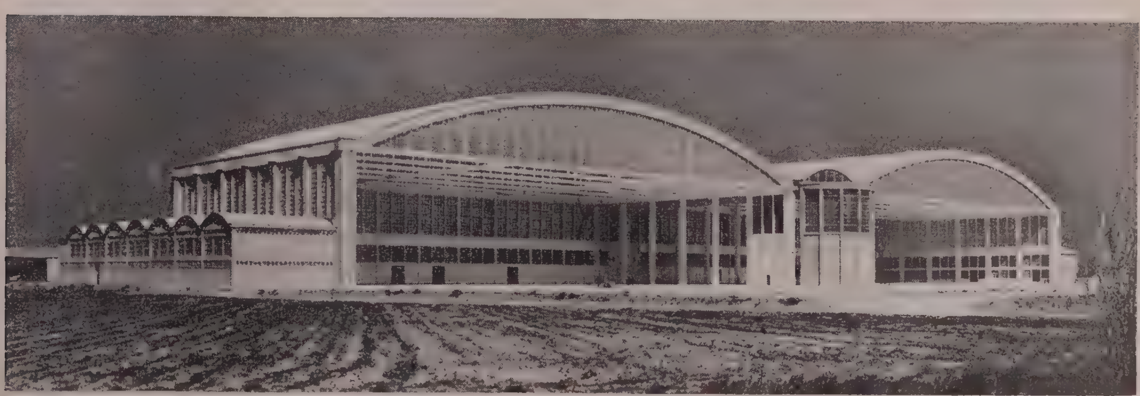

Fig. 14.—The hangar at Marignane Airport. Exterior view

The springings are shaped in a special way which solves the difficulty.

In view of its spatial nature, the structure was so complex in character that the strains and stresses were tested on a one-to-five scale model which gave accurate results.

The quantity of concrete in the roof is equivalent to a uniform thickness of less than $4\frac{1}{2}$ inches distributed

This extreme lightness in the weight of the structure does not impair its safety ; the stresses remain moderate and fluctuate between 640 and 930 lb./sq. inch.

The tie-beams are braced along a third of their span by horizontal struts which transmit the wind stresses from the gable end to the diagonal struts acting as buttresses at the rear. The maximum thrust of 400 tons on each tie-beam is absorbed, with a factor of safety of two, by

means of 208 six millimetre high tensile steel bars (about $\frac{1}{4}$ inch) having a breaking stress of 200,000 lb./sq. inch.

Each tie-beam was hollow in section to contain its reinforcement which was stressed by jacks. An amount of pre-compression was imposed upon the concrete section of the tie-beam to allow for snow load, and this was followed by injection of cement grout.

A fundamental question in roofs of this magnitude is that of the general stability of the whole of the structure together with freedom of expansion and contraction. According to the direction of the wind, the horizontal forces are all absorbed by, either eight diagonal struts at the rear, or by two spandrils walls parallel to the gables and placed in the inter-bay, between the two bays of the hangar.

To allow for horizontal movement, the inter-bay columns are knee-jointed at the top, and the side columns knee-jointed top and bottom, thus acting as 28 ft.-high pendulums. The joints are in ferro-concrete and carry a permanent point load of 230 tons.

(II) Architecture

From an architectural point of view it is noteworthy that Maitre Auguste Perret brought a good deal of aesthetic improvement, whilst, at the same time, maintaining the original structure as designed by the Civil Engineers, but every item was carefully examined. He improved on the anchorage of the tie-beams, on the diagonal struts at the rear and suggested that the out-sheds built on the side of the main building should be placed perpendicular to the gables instead of parallel with the facade. He evolved the square pattern of the window frames to the human scale of vision so as to bring out in the open the monumental size of the hangar proper. (Fig. 14.)

(III) Construction of the Roof

Each of the two bays of the hangar weighed four thousand tons including the gables and spread over an area of 70,000 sq. ft. To our knowledge, the raising of a structure of that size is without precedent. This was not a technical fancy. The method of construction was decided upon after careful examination of all the factors involved.

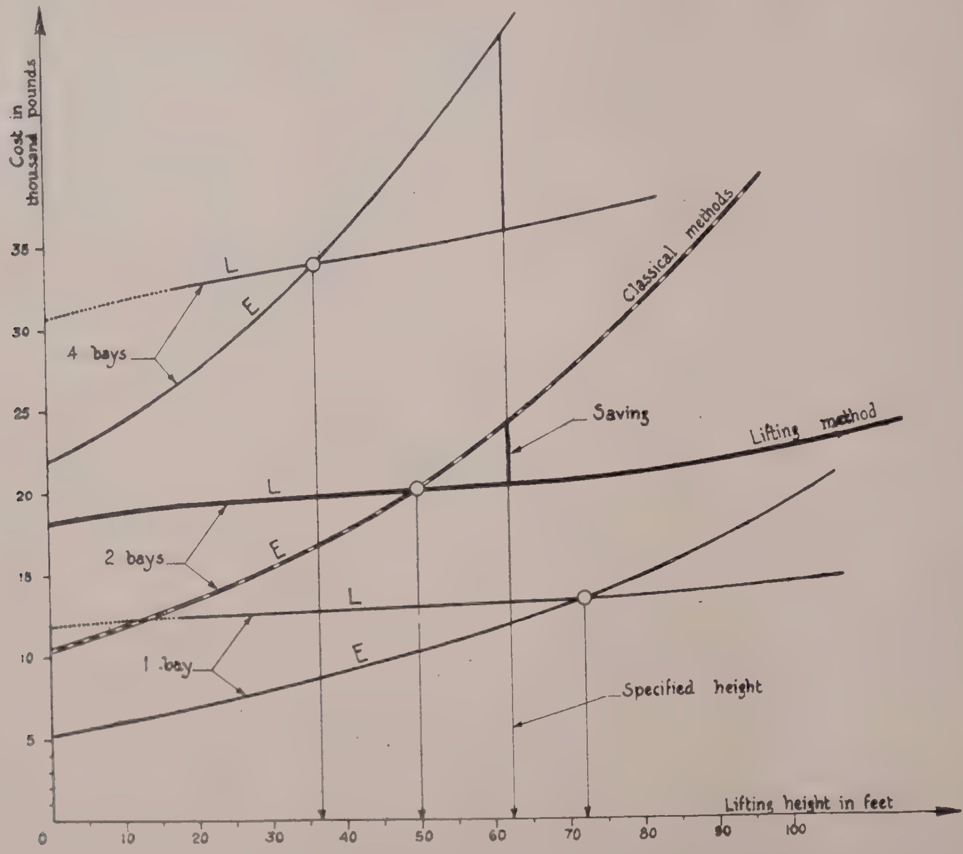

Fig. 15.—The hangar at Marignane Airport. Curves of comparative costs

Various alternatives were considered and curves of comparative prices were plotted. (Fig. 15.) The resultant curve *E* corresponds to the classical method of erection with the usual type of scaffolding as high as the structure itself, passing from one bay to the other and used twelve times over.

The curve *L* refers to two classical types of scaffolding as high as the rise of the roof only and resting at ground level plus the lifting of the completed roof to the specified level. The scaffolding is used six times over.

In the case of two bays, which is the one under construction, curves *E* and *L* intersect when the lift is 50 feet where the costs are the same. In the present case, the lift is 62 ft. and there is a definite saving in cost when the jacking method is applied. This has now been confirmed long after completion of the work.

It is interesting to note that these comparative curves show that the jacking method would have been more

expensive in the case of one bay only, but considerably cheaper in the case of four bays.

To carry out the jacking method successfully both technically and economically, we had decided to respect the following rules :—

(1) Maintain the stresses centred along their final axis.

(2) Ensure free expansion and contraction during lifting process.

(3) During the whole of the operations, maintain a factor of safety at least equal that of the final structure.

(4) Use current site equipment and avoid expensive mechanical appliances.

(5) Avoid temporary lifting members or wind bracings which would not be retained in the final structure.

Consequently, the jacks take bearing upon the permanent 40-inch-square columns made up of prefabricated elements 42 inches long and comprising U-shape exterior elements and rectangular central elements.

During the lifting operation, the elements were placed one on top of another, without grouting, and tied together by means of corner rods suitably threaded.

The open face of the U-shaped element was vacuum concreted at the end of each day, and all remaining hollow parts filled in by cement grout injections.

A 300-ton hydraulic jack was applied to each column. The jacks were anchored in the roof with the piston directed downwards, and fitted with four return springs to bring back the piston to its original position.

After each lift, concrete spacing blocks were inserted on the side to hold the roof in its new position. The jacks were then released and the pistons sprung up to their starting position. This left a gap which was immediately filled by a $7\frac{1}{2}$-inch thick concrete packing block. (Fig. 16.)

After five packing blocks had been inserted, the space available at the next lift was sufficient to receive the 42-inch-long U-shaped element. The blocks were then removed and the 42-inch-long rectangular central element placed into position. The whole operation lasted but half a day.

During the lifting operations, the roof was guided along its correct alignment by clamping it temporarily to the inter-bay columns in one direction, and to the vertical columns of the rear buttresses, in the other direction perpendicular to the first. Consequently, no horizontal movement could possibly take place.

These clamps were fitted with shoes sliding along removable guide rails bolted on to the columns. The guide rails were short lengths and were used over and over again as the lifting operation proceeded on its course.

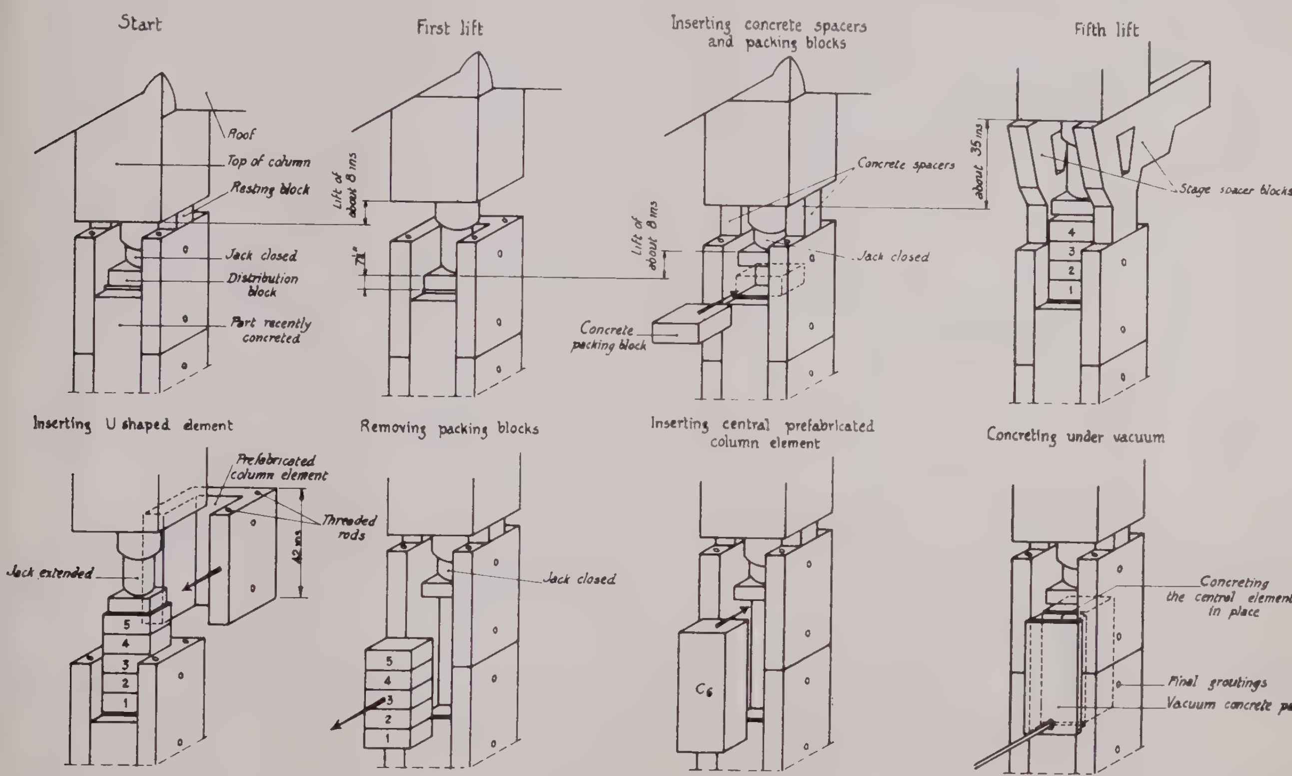

Fig. 16.—The hangar at Marignane Airport. Various stages during lifting operations

Warning lights were used at each jack and instructions were co-ordinated by telephone from a central control board.

The total power used to raise the whole of the roof never exceeded 16 h.p., and the lifting operation was carried out at the rate of two feet an hour.

I should like to conclude by bringing to your attention some of the advantages of this method of construction which I have considered noteworthy. They are as follows :—

1. Any work carried out at ground level is much safer and more accurate than on high scaffoldings. Ordinary lifting tackle may currently be used.

2. An improvement in the efficiency of the men who feel safer when they are working almost at ground level, and where, incidentally, supervision of the work is easier.

3. Finally, work at ground level affords the possibility of full-size structural tests without danger and consequently, lends itself best to the execution of the most ambitious of large-scale undertakings.

(Translated by P. GERARD.)

Institution Notices and Proceedings

FORTHCOMING MEETINGS

The following meetings will be held at 11, Upper Belgrave Street, London, S.W.1.

Thursday, November 11th, 1954

Joint Meeting with the British Section of the Société des Ingénieurs Civils de France at 6 p.m., when Monsieur N. Esquillan, M.Soc.C.E.(France), will give a paper on "Examples of Precast Ferro-Concrete Constructions in France."

Thursday, November 25th, 1954

Ordinary General Meeting for the election of members, at 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when Mr. D. R. R. Dick, B.Sc., M.I.C.E., will give a paper on "The Design and Construction of the Nuclear Reactor Buildings at Windscale Works, Sellafield."

Thursday, December 16th, 1954

Ordinary General Meeting, 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when Mr. M. F. Palmer, M.I.C.E., M.I.Struct.E., will give a paper on "Fabrication and Erection of Steel Plate Girder Railway Bridges."

Thursday, January 13th, 1955

Ordinary Meeting, 6 p.m., when Mr. W. R. Garrett, A.M.I.C.E., A.M.I.Struct.E. (Associate Member of Council), will give a paper on "Gasholder Development and Design."

Thursday, January 27th, 1955

Ordinary General Meeting, 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when Mr. G. M. Boyd, A.M.I.Struct.E., M.I.N.A., will give a paper on "Brittle Fracture Problems in Steel Construction."

Members wishing to bring guests to the Ordinary and Joint Meetings referred to above, are requested to apply to the Secretary for tickets of admission.

EXAMINATIONS—JANUARY, 1955

The Examinations of the Institution will next be held at centres in the United Kingdom and Overseas on January 11th and 12th, 1955 (Graduateship), and January 13th and 14th (Associate-Membership).

LONDON GRADUATES' AND STUDENTS' SECTION

The following meeting will be held at 11, Upper Belgrave Street, London, S.W.1:—

Tuesday, November 23rd, 1954

Members of the Section are invited to give short talks of 15-20 minutes' duration on various aspects of structural engineering. A prize will be awarded for the most meritorious contribution. Will those wishing to contribute please advise the Honorary Secretary, J. A. Pope, 53, Cranleigh Drive, Leigh-on-Sea, Essex.

Wednesday, January 19th, 1955

Annual Dance.

BRANCH NOTICES

LANCASHIRE AND CHESHIRE BRANCH

The following meetings have been arranged :—

Monday, November 29th, 1954

At the College of Technology, Manchester, at 6.30 p.m. Mr. S. M. Cooper, A.M.I.Struct.E., on "The Design ...res and Collapse Investigation of the Tacoma ...ws Bridge" (with the film).

Tuesday, November 30th, 1954

The above meeting will be repeated in Liverpool.

Tuesday, December 7th, 1954

Trip to John Summers & Sons' Steelworks, Shotton, by the Liverpool members of the Branch.

Friday, January 7th, 1955

At the College of Technology, Manchester, 6.30 p.m. Dr. P. W. Rowe, B.Sc., A.M.I.C.E. (Graduate), on "The Present Situation on Retaining Wall Design."

Tuesday, January 11th, 1955

A Joint Meeting with the Institute of Welding (Liverpool and District Branch) will be held at the Liverpool City College of Technology, Liverpool, at 7 p.m., when Mr. L. Rotherham will give a paper on "Welding at the Atomic Plants."

Tuesday, January 25th, 1955

At the College of Technology, Manchester, 6.30 p.m. Dr. T. Howarth, A.R.I.B.A., on "Modern Architecture."

Joint Hon. Secretaries : A. S. Sinclair, A.M.I.Struct.E., 17, The Circuit, Cheadle Hulme, Cheshire ; M. D. Woods, A.M.I.Struct.E., 58, Spring Gardens, Salford.

MIDLAND COUNTIES BRANCH

The following meetings have been arranged :—

Thursday, November 4th, 1954

At the Public Library, Stafford, 7 p.m., Mr. Edgar Morton, M.Sc., P.A.Inst.W.E., Hon.M.I.Q., on "Examples of Site Exploration."

Tuesday, November 16th, 1954

At the Supper Room, The King's Hall, Queen Street, Derby, 7.0 p.m., Mr. E. Williams on "The Construction of a Prestressed Concrete Gas Holder Tank at Cromer."

Friday, November 26th, 1954

Joint Meeting with the Birmingham and Five Counties Architectural Association, at the James Watt Memorial Institute, Birmingham, 6 p.m., Mr. S. Woolf on "Structural Use of Timber."

Tuesday, January 4th, 1955

Joint Meeting with the Reinforced Concrete Association, Midland Counties Branch, at the Midland Institute, Birmingham, 6 p.m., Dr. F. G. Thomas, B.Sc., M.I.C.E., M.I.Struct.E. (Member of Council), on "Load Factor Methods of Design of Reinforced Concrete."

Friday, January 28th, 1955

Joint Meeting with the Institute of Welding, Midland Section, at the James Watt Memorial Institute, Great Charles Street, Birmingham, at 6 p.m., Mr. S. M. Reisser, B.Sc., M.I.Struct.E., A.M.I.C.E., will give a paper on "The Influence of Welding on Steel Building Structures with Particular Reference to Erection."

Hon. Secretary : L. A. Firminger, A.M.I.Struct.E., 656, Chester Road, Erdington, Birmingham, 23.

MIDLAND COUNTIES GRADUATES' AND STUDENTS' SECTION

The following meetings have been arranged :—

Tuesday, November 30th, 1954

Details will be circulated.

Monday, January 1st, 1955

Joint Meeting with the Junior Section of the Birmingham and Five Counties Architectural Association, at the James Watt Memorial Institute, Great Charles Street, Birmingham, at 6 p.m., Mr. H. V. Hill, M.Sc., A.M.I.C.E.,

A.M.I.Struct.E. (Associate-Member of Council), will give a paper on " The Use of Light Alloys in Structures."

Hon. Secretary : H. L. Bramwell, 139, Wood Lane, Handsworth Wood, Birmingham, 20.

NORTHERN COUNTIES BRANCH

The following meetings have been arranged :—

Tuesday, November 2nd, 1954

At the Cleveland Scientific and Technical Institution, Corporation Road, Middlesbrough, at 6.30 p.m., Mr. G. Little, M.Sc., will give a paper on " Further Studies of Load Distribution in Bridge Decks."

Wednesday, November 3rd, 1954

The above meeting will be repeated at the Neville Hall, Westgate Road, Newcastle upon Tyne, at 6.30 p.m.

Wednesday, December 1st, 1954

At the Neville Hall, Westgate Road, Newcastle upon Tyne, at 6.30 p.m., Mr. A. P. Clarke, B.Sc., will give a paper on " Lackenby Steelworks."

Tuesday, December 7th, 1954

At the Cleveland Scientific and Technical Institution, Corporation Road, Middlesbrough, at 6.30 p.m., Mr. E. Czeiler, M.I.Struct.E., will give a paper on " Steelwork for Hindhaugh Street Flats, Newcastle."

Wednesday, January 12th, 1955

Joint Meeting with The Institution of Civil Engineers, at the Cleveland Scientific and Technical Institution, Corporation Road, Middlesbrough, at 6.30 p.m., when Dr. A. W. Skempton, A.M.I.C.E., will give a paper on " Some Practical Applications of Soil Mechanics."

Thursday, January 13th, 1955

Joint Meeting with the Northern Architectural Association, at Higham Place, Newcastle, at 7.30 p.m.

Hon. Secretary : Captain O. Lithgow, A.M.I.Struct.E., 4, Stoneleigh Avenue, Acklam, Middlesbrough.

NORTHERN IRELAND BRANCH

The following meetings have been arranged :—

Tuesday, November 2nd, 1954

Annual Dinner and Social Function, at the Grand Central Hotel, Belfast, at 6.30 p.m. Visit of the President and the Secretary of the Institution.

Tuesday, December 7th, 1954

At the College of Technology, Belfast, at 6.45 p.m., Mr. W. S. Atkins, B.Sc., M.I.C.E., M.Inst.W., will give a paper on " Structural Steelwork and Concrete Construction, with Particular Reference to Abbey Steelworks."

Tuesday, January 4th, 1955

At the College of Technology, Belfast, at 6.45 p.m., Mr. J. D. Boyd, B.Sc., will give a paper on " Reinforced Concrete Portal Frames in Northern Ireland."

Hon. Secretary : A. H. K. Roberts, B.A., B.A.I., M.I.Struct.E., M.I.C.E., " Barbizon," 26, Dunlambert Park, Belfast.

SCOTTISH BRANCH

The following meetings have been arranged :—

Wednesday, November 17th, 1954

Joint Meeting with the West of Scotland Branch of The Institute of Welding, at the Institution of Engineers and Shipbuilders, 39, Elmbank Crescent, Glasgow, at 7 p.m., when Mr. S. M. Reisser, B.Sc.(Eng.), M.I.Struct.E., A.M.I.C.E., will give a paper on " The Influence of Welding on Steel Building Structures."

Friday, December 3rd, 1954

Joint Meeting with the East of Scotland Branches of The Institute of Welding and the Society of Engineers, at the Heriot Watt College, Chambers Street, Edinburgh. Paper on " Welded Structures."

Tuesday, January 18th, 1955

At the Institution of Engineers and Shipbuilders, 39, Elmbank Crescent, Glasgow, at 7 p.m., Mr. Hugh Fraser, B.Sc., M.I.Struct.E., A.M.I.C.E., and Mr. W. G. Cantlay, B.Sc., A.M.I.C.E., A.M.I.Struct.E., will give a paper on " A Graphical Approach to the Design of Two and Three Span Rigid Frame Buildings."

Hon. Secretary : G. Drysdale, A.M.I.Struct.E., " Niaroo," 33, Union Street, Motherwell, Lanarkshire.

SOUTH-WESTERN COUNTIES BRANCH

The opening meeting of the Session will be held at the Duke of Cornwall Hotel, Plymouth, on Saturday, November 13th, at 6.30 p.m., when the Chairman's Address will be given by Colonel R. Hazzledine, O.B.E., T.D., M.I.Struct.E. The President and the Secretary of the Institution will attend the meeting, which will be followed by an informal dinner.

Joint Hon. Secretaries : E. W. Howells, M.I.Struct.E., 10-12, Market Street, Torquay ; C. J. Woodrow, 23, Torland Road, Hartley, Plymouth.

WALES AND MONMOUTHSHIRE BRANCH

The following meetings have been arranged :—

Tuesday, November 2nd, 1954

At the Mackworth Hotel, Swansea, at 6.30 p.m., Mr. F. V. M. Bell, will repeat his Chairman's Address which will be followed by a discussion.

Saturday, November 6th, 1954

At the County Buildings, Colwyn Bay, at 6 p.m., the Chairman's Address will be repeated and will be followed by a discussion.

Tuesday, November 16th, 1954

Joint Meeting with the Institute of Welding, at the South Wales Institute of Engineers, Park Place, Cardiff, at 6.30 p.m., when Mr. F. Brooksbank, M.A.(Cantab.), (Graduate), will give a paper on " Economics in Welding Design."

Wednesday, December 8th, 1954

At the Mackworth Hotel, Swansea, at 6.30 p.m., Mr. H. E. Lewis, B.Sc., D.I.C. (Graduate), will give a paper on " Developments in the Structural Use of Concrete."

Monday, January 31st, 1955

Joint Meeting with the Institution of Civil Engineers, at the South Wales Institute of Engineers, Park Place, Cardiff, at 6.30 p.m., when Dr. B. G. Neal, A.M.I.C.E., will give a paper on " Simple Plastic Theory."

Hon. Secretary : K. J. Stewart, A.M.I.C.E., A.M.I.Struct.E., 15, Glanmor Road, Swansea.

WESTERN COUNTIES BRANCH

The following meetings have been arranged :—

Friday, November 12th, 1954

Combined meeting with the Institution of Civil Engineers, at the University of Bristol Geology Lecture Theatre, at 6 p.m., when Mr. H. C. Husband, B.Eng., M.I.C.E., M.I.Mech.E., M.I.Struct.E. (Honorary Librarian), will give a paper on " Unusual Industrial Structures."

Friday, December 3rd, 1954

At the University of Bristol Geology Lecture Theatre, at 6 p.m., Mr. J. Guthrie Brown, M.I.C.E., M.I.Struct.E. (Vice-President), will give a paper on " Highlights in an Engineer's Life."

Friday, December 10th, 1954

Combined Dance, Royal Hotel, Bristol.

Friday, January 7th, 1955

At the University of Bristol Geology Lecture Theatre, at 6 p.m., Mr. H. G. Lakeman, A.C.G.I., B.Sc., Eng. (London), A.M.I.C.E., will give a paper on "Recent Bridge Works in Bristol District."

Hon. Secretary : E. Hughes, A.M.I.Struct.E., 23, Southdown Road, Westbury-on-Trym, Bristol, 9.

YORKSHIRE BRANCH

The following meetings have been arranged :—

Wednesday, November 17th, 1954

At the Great Northern Hotel, Leeds, at 6.30 p.m., Mr. D. V. Pike, M.I.Struct.E., A.M.I.C.E., will give a paper on "Aluminium Alloy Structures."

Wednesday, December 15th, 1954

At the Great Northern Hotel, Leeds, at 6.30 p.m., Mr. A. P. Clark and Mr. T. V. Thompson, M.I.Struct.E., will give a paper on "Lackenby Steelworks."

Wednesday, January 19th, 1955

At the Great Northern Hotel, Leeds, at 6.30 p.m., Mr. F. A. Charman, B.Sc., A.M.I.C.E., will give a paper on "The Construction of The Woodhead New Tunnel."

Hon. Secretary : E. Wrigley, A.M.I.Struct.E., 17, The Drive, Alwoodley, Leeds.

UNION OF SOUTH AFRICA BRANCH

Branch Hon. Secretary : A. E. Tait, B.Sc., A.M.I.C.E. P.O. Box 3306, Johannesburg, South Africa. During week-days Mr. Tait can be contacted in the City Engineer's Department, Town Hall, Johannesburg. Phone : 34-1111, Ext. 257.

Natal Section Hon. Secretary : E. G. Bennett, A.M.I.Struct.E., c/o The Reinforcing Steel Co., Ltd., P.O. Box 49, Merebank, Durban.

Cape Section Hon. Secretary : R. Stubbs, M.I.Struct.E., The Reinforcing Steel Co., Ltd., P.O. Box 2692, Cape Town.

ADDITIONS TO THE LIBRARY

ABBETT, R. W. *Engineering Contracts and Specifications.* 3rd Edition. New York and London, 1954.

Aluminium Development Association Symposium on Aluminium in Building, June, 1953. London, 1954.

BRAITHWAITE & CO. ENGINEERS, LTD. 215 *Photographs of Structural Steelwork of all types.* London, 1954.

British Iron and Steel Research Association. *Researches of the Corrosion Committee—The Prevention of Corrosion.* London, 1954.

British Iron and Steel Research Association Annual Report, 1953. London, 1954.

British Standards Institution Yearbook, London, 1954.

CAUGHEY, R. A., and CEBULA, R. S. *Constants for Design of Continuous Girders with abrupt changes in Moments of Inertia.* Iowa, U.S.A., 1954.

CHARLTON, T. M. *Model Analysis of Structures.* London, 1954. Presented by Mr. R. J. Cornish.

DISNEY, L. A. *Steel Reinforcement.* London, 1954.

Dorman Long & Co., Ltd. *Lackenby Open Hearth Steelworks : The Principles Behind the Project.* Middlesbrough, 1954.

Federation of British Industries Register of British Manufacturers. 26th Edition. London, 1954.

GRAVINA, P. B. J. *Teoria de Calculo das Pontes Pênseis.* Brazil, São Paulo, 1951.

Imperial Chemical Industries Engineering Codes and Regulations. Design, Layout and Installation of Machines ; Safety Precautions. London : Royal Society for Prevention of Accidents, 1954.

Institution of Civil Engineers Committee on Engineering Bills of Quantity. Standard Method of Measurement of Civil Engineering Quantities. London, 1953.

Institution of Mechanical Engineers Proceedings (B), Nos. 1-12, 1952-3, Vol. IB. London, 1954.

KLEINLOGEL, A. *Baustoffverarbeitung und Baustellenprüfung des Betons.* 2nd Edition, Berlin, 1951.

LAIR, E. A. *Carpentry Workbook.* New York and London, 1954.

MAGNEL, G. *Prestressed Concrete.* 3rd Edition, London, 1954.

MOUAT, G. D. *Notes and Nomographs for the Design of Flat Rectangular Plates under Normal Pressure.* Richmond, Surrey, 1954.

NAUNTON, W. J. S. *What Every Engineer should know about Rubber.* London, 1954.

OSTLUND, L. *Lateral Stability of Bridge Arches braced with Transverse Bars and Appendix to Chapter IV : Calculation of Critical Load on various assumptions in two examples.* Göteborg, Sweden, 1954.

PENNINGTON, A. M. *Concrete Farm Structures.* London, 1954. Presented by Mr. R. J. Vernall.

PETTERSSON, D. O. *Circular Plates subjected to Radially Symmetrical Transverse Load Combined with Uniform Compression or Tension in the Plane of the Plate.* Stockholm, 1954.

REA, J. T. *Appendix to the 11th Edition of "How to Estimate for Building Work."* Compiled by A. E. Baylis. London, 1953.

Revista de la Real Academia de Ciencias Exactas Fisicas y Naturalas, Vol. XLVII. Madrid, 1953.

TERRINGTON, J. S. *Girder Flange Design.* Revised Edition. London, 1953.

TERRINGTON, J. S., and HAWKES, J. M. *The Design of Crane Gantry Girders for Steelworks.* Revised Edition. London, 1953.

YOUNG, J. McHardy. *Structural Theory and Design.* One-Volume Edition. London, 1954.

DRURY MEDAL AWARD

The fifth competition for the above award will take place in 1955. The subject is the design of a mobile crane.

Graduates and Students of the Institution who wish to compete are invited to apply for full details to the Secretary : envelopes to be marked in the top left-hand corner, "Drury Medal Award."'

The closing date for the competition is October 1st, 1955.

The general conditions of the competition are as follows :—

1. The competition shall be for Graduates and Students of the Institution of not more than 25 years of age.

2. The subject of the competition shall be a design of structural character, that is to say, primarily structural design, not planning.

3. The subject of design and conditions shall be prepared and issued biennially by a group of five members appointed by the Council.

4. The Literature Committee shall appoint a Jury of not less than five to examine the works submitted and to interview candidates, if found necessary.

5. In order to show that the work submitted is solely the work of the competitor, the documents submitted shall be countersigned by a corporate member of the Institution, or failing this, shall be accompanied by a declaration on a prescribed form signed by the candidate in the presence of a Justice of the Peace or a Commissioner for Oaths.

Presidential Address*

THE DEVELOPMENT OF STRUCTURAL ENGINEERING AS A PROFESSION

By S. B. Hamilton, Ph.D., M.Sc., B.Sc.(Eng.), M.I.C.E., M.I.Struct.E.

To be called upon to deliver a Presidential Address to the Institution of Structural Engineers is a high privilege and a considerable responsibility. It is a source of pride and gratitude to find oneself elected by members of one's chosen calling to the highest honour they can bestow ; but at the same time of humility, when one compares the contributions of one's predecessors to the advancement of the science and art of Structural Engineering with one's own modest achievements. Perhaps on such an occasion it is permissible to recall the early days of one's association with the Institution.

Thirty years ago I had recently entered a Government drawing office. My work was often interesting and sometimes important ; but compared with the frequently excessive personal responsibility which, on quite inadequate previous experience, I had had to assume as a contractor's engineer abroad, the protracted deliberations before the Service hierarchy made decisions, though doubtless proper in the conduct of works at the public expense, seemed likely to pen one into a somewhat narrow world. To keep in touch with the wider world outside the office I did two things : volunteered to serve on Institution Committees, where I could meet engineers who worked outside the sheltered fold : and took up evening teaching, which compelled me to face structural problems in their broadest form, and to explain them as simply and directly as I could in terms of fundamental principles.

My first work for the Institution was on the Masonry Sectional Committee, which adopted a suggestion of mine for a piece of research work, and put me in charge of a panel to get the work done. After a few years I was elected Chairman of the Sectional Committee, only to find that, as such I must also serve on the Science, Literature and Legislation Committees. Since then I have served on many Committees, been Chairman of several, and so, through membership of Council and through various honorary offices, to the Presidential Chair. The fellowship of the active inner core of the Institution has always been to me a source of pride and pleasure. Not the least among my interests has been the work of the Education and Examinations Committee. It has always been the aim of that Committee and of its Moderating sub-Committee to see that the standard of the examinations was maintained at a level which should challenge the able and ambitious student to exert his best efforts ; but should never become a mere endurance test negotiable only by the exceptionally persistent. The large number of entries, increasing with every year ; and the high quality of the work submitted by the successful candidates give good cause to believe that by and large the Committee has achieved its aim, and that between them the Education and Examinations, and the Membership Committees have good reason to believe that the new entrants they recommend for every class of membership are worthy to take their place in a truly professional calling.

In these days the title " professional " is widely and sometimes loosely used. It might, therefore, be well to make clear in what sense it is here employed. The earliest professions were the Church, the Law and Medicine. All the well-established bodies, which claim the title of professional men for their members, have certain features in common with the ancient professions.[1]

The status of the professional body is guaranteed by a Royal Charter or by legal Incorporation. A substantial proportion of its members act towards their clients in a " fiduciary capacity," i.e., the member is the trusted personal adviser of his client. The professional body itself lays down the conditions of admission to its ranks, and administers to entrants such qualifying tests as its members consider necessary. The professional body demands that those it admits shall maintain a high standard of conduct and reserves the right to discipline any member whose conduct falls below that standard. Most professional bodies also confer on their members a title and letters designate which distinguish them from unrecognised practitioners, if such there be. In all these respects Structural Engineering, as defined in the Institution's Charter, is a profession.

The most characteristic of these features is the personal independence of the professional man and the strictly fiduciary relationship between himself and his client. Even in the oldest professions, however, this personal independence is severely restricted for that increasing number of members who are not in private practice but who hold appointments under the State, some other public body, a business, or a manufacturing firm. So long as employed persons comprise a minority of the membership, and some do and all could (at least in theory) change from employment into private practice and vice versa; so long, also, as both practitioners and employees receive the same training and pass the same examinations, it would be invidious to distinguish between them. But if the qualities and characteristics which have hitherto given the professions their honourable status are to be maintained, the professional bodies may have to guard very carefully such rights to individuality as still remain to their employed members. Apart from the dilution of professional status through its widening to include new specialist callings, the essential character of the professions is bound to change, and could indeed be lost in the general tendency of everything to come under centralised political control, even in those states whose citizens still pride themselves on their freedom. It was, however, as servants of the French Crown that engineers were first accorded a recognised status. It came about in the following way.

The First Professional Engineers

In seventeenth-century France, industry and commerce were sorely hampered by inadequate means of transport. To improve matters, with a view to increasing taxable wealth, Jean Baptiste Colbert (1619-1683), Chancellor to King Louis XIV, appointed in each généralité an engineer responsible to himself (Colbert) for the conduct of public works, particularly

Given before a General Meeting of the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1, on Thursday, October 7th, 1954, at 6.0 p.m.

highways and canals. He also persuaded the King to establish in 1666 the Académie Royale des Sciences (somewhat on the model of the Royal Society, chartered in 1662) : and in 1671 the Académie de l'Architecture. The latter body gave permanence to a consultative committee of practising architects, who were superintending the building of the Louvre. They were expected thenceforth to teach as well ; but do not seem to have taken that side of their duties very seriously. A decree of 1676 which forbade the use of the title "Architect" to any but members of the new Académie left no option to other craftsmen but to work on contract under the recognised practitioner, as, indeed, had been the practice at the Louvre since about 1600. Henceforth contract work became general, usually for the work of one trade done to specification and paid as measured work on a schedule of prices. Hitherto there had been no clear demarcation between the Architect, the Civil Engineer, the Military Engineer, and the Contractor. Indeed, in Renaissance Italy and France one man might serve on occasion in all four capacities. The Corps du Génie became a properly constituted branch of the French Army in 1672, under Vauban, who had trained and organised it.

By the end of the eighteenth century in France official building (and there was little other of importance) had come under the control of Jules Hardouin Mansard (1645 ?-1708) who from 1675 had directed the building of the Palace of Versailles. Shortly before he died Mansard designed two bridges—at Blois and at Moulins—both of which collapsed shortly afterwards. These mishaps and the sad neglect which had befallen the highways since Colbert's organisation had been allowed to decline, led in 1715 to the appointment of an Inspecteur Général des Ponts et Chaussées, and in 1716 to the organisation under him of a number of architects and others serving on *ad hoc* appointments, or well-known for their skill in construction. There was, however, no provision for training juniors. In 1747 a Registry of Plans was formed with three draughtsmen to look after it. By 1750 the three had grown to thirteen. The establishment was re-named the Ecole des Ponts et Chaussées. At the same time the Corps was reorganised, both Corps and school being placed under the directorate of Jean Rodolphe Perronet (1708-1794). The school became the first successful training centre for Civil Engineers ; but it had no regular teachers. Instruction was given either by engineers, or by pupils with special knowledge, verbally or by means of lithographed notes.

In 1794 the Ecole des Ponts et Chaussées with other specialist training centres was merged in the Ecole Centrale des Travaux Publics, an unwieldy body which in 1795 was split into the Ecole Polytechnique which received all new entrants, and the senior specialist schools, which included the re-founded Ecole des Ponts et Chaussées.

Gaspard Monge (1746-1818) as head of the Ecole Polytechnique introduced three novel features : open competitive entrance examinations, lectures to large classes by leading scientists appointed as whole-time teachers, and practical work by students in physical and chemical laboratories. This general set-up was adopted by the German Polytechnic Institutes, founded after the Napoleonic Wars ; and considerably later in the faculties of Science and Technology in British Universities, [illegible] that neither in Germany nor in Britain was the State responsible for running the schools or appointing the staffs. Apart from the schools founded as above to train engineers and others for Government employment the Convention of 1794 decreed the foundation of the Conservatoire National des Arts et Métiers to serve both as an industrial museum and a teaching centre for members of the public, more particularly craftsmen. A scientific training for technologists outside the Government service was not made available until 1829 when the Ecole Centrale des Arts et Manufactures was founded as a private venture. In 1860 this school was taken over by the State, but without changing its function.

Developments in Britain

In contrast with its development in France, Civil Engineering in Britain was never organised by the State ; indeed it was hardly organised at all until the nineteenth century. Works could only be done by small groups of "adventurers" at their own expense, under powers granted to them by Letters Patent. Such works consisted first of improvements to the navigable rivers to extend the length accessible to coastal shipping, thus to shorten the haul by expensive pack-horse carriage inland from the ports. Next came works of drainage to bring low-lying and potentially fertile land into cultivation. The greatest of these drainage works was that undertaken in the Fens by Cornelius Vermuyden (c. 1596-c. 1683). For other works of novelty and unusual magnitude foreigners were employed. For instance : Tide mills to work pumps to raise river water for distribution through mains consisting of bored elm trunks and service pipes of lead were installed first in Britain under the arches of London Bridge in 1582 by Peter Morice—a German. The London wheels and pumps were re-built in 1704 by George Sorocold, of Derby, who erected similar installations in about a dozen other towns. Sorocold used cast-iron for his crank-shafts, pistons and pump-barrels —the first recorded instance of the use of that material in English machine construction. He also constructed the first large dock—the Howland Great Dock in Rotherhithe—and advised upon the first dock for Liverpool, although that actually made was the work of Thomas Steers (d. 1750), who had been Sorocold's assistant at Rotherhithe.[2]

The last important civil engineering work in eighteenth century Britain to be given to a foreigner was the design and erection of Westminster Bridge in the seventeen-thirties by Charles Labelye (1705-81). This appointment aroused fierce criticism from disappointed competitors. Thenceforth, particularly in mechanical engineering, it was the Briton who went abroad, either to sell his wares or to set up works.

An important change in the choice of Building Materials in England began in the re-building of London after the Great Fire of 1666. Hardwood timber suitable for framing was becoming scarce and, being combustible, its use in external walls was forbidden. Brick, hitherto little used in London except for paving, became the general material for walls, and imported softwood for floors and roof-work. Building tradesmen from far and wide came to work in the re-building of London. Many returned later carrying back with them the new practices devised to comply with the Re-building Acts. The lime used in mortars was usually that from the nearest natural source. The best lime was supposed to be from the hardest stone, marble yielding the best of all. Actually there was no truth in this supposition : marble is purer than most natural limestones ; its hardness is irrelevant. Some limes had the property of setting under water. These were termed "hydraulic limes" ; but the reason was not known until John Smeaton (1724-92) discovered it in the course of his search for the best lime to use in his work on the Eddystone Lighthouse. He collected samples of every available limestone, making experiments with the lime obtained from them.

He thereby discovered that every hydraulic lime was the product of burning a limestone which also contained clay.

Eddystone made for Smeaton a reputation which brought him many enquiries and requests for reports ; but, because of the difficulty in those days of financing public works, few schemes got beyond the project stage. His reports, however, published in four volumes by the Royal Society (1797-1812) are models of what an engineer's report should be.

Smeaton took an active part in the foundation in 1771 of the Society of Civil Engineers, although he was never its President. Its chief purpose was to bring together those engineers who were likely to be called upon to give evidence before Parliamentary Committees on the merits of rival canal schemes and the like, in order that they might have the opportunity of discussing differences of technical opinion in a friendly way rather than make their first contacts in public and with acrimony, as the supporters of conflicting commercial interests. By 1818 the Society had become somewhat exclusive, and the Institution of Civil Engineers was founded on a wider basis. The Society still exists as the "Smeatonian" ; but now only functions as a dining society.

Thomas Telford (1757-1834), President of the Institution of Civil Engineers from its foundation until his death, began life as a working mason ; his famous contemporary John Rennie (1761-1821) as a millwright. While learning his craft, however, Rennie had attended classes at Edinburgh University, although he did not work for a degree. It was in carrying out works designed by Telford, Rennie and a few others that firms of public works contractors came into being in England, as they had done in France a century earlier.

Nothing developed in England in any way comparable with the Ecole des Ponts et Chaussées. At Edinburgh Professor John Robison (1739-1805) introduced a course of Mechanical Philosophy : and at Cambridge some lectures on Engineering were given from 1796 under a wide interpretation of the duties of the Jacksonian Professorship. But these were merely side-lines to courses of study predominantly classical. In the early nineteenth century a young man aspiring to become a civil engineer could do so only by entering the office of one already established, and could study only from French text-books. Both Rennie and Telford had taught themselves French for that very purpose ; so did Thomas Tredgold.

In 1800 Benjamin Thompson, Count Rumford (1753-1814) proposed the foundation in London of a Royal Institution to run on much the same lines as the Conservatoire National des Arts et Métiers in Paris. The Royal Institution was duly founded, but with funds inadequate to fulfil any such purpose ; and Humphrey Davy, as resident lecturer, addressed his discourses not to mechanics, but to men and women of fashion, and to manufacturers able to finance research.

Rumford's intention was in part met by the Mechanics' Institutes, which were founded on the bequest of John Anderson (1726-1796), Professor of Natural Philosophy in Glasgow University, who had himself lectured to artisans from 1760. His work was carried on by Dr. Birkbeck, who lectured at Anderson's Institution until 1804, and then started a similar movement in London. Mechanics' Institutions were opened in Glasgow and in London in 1823, in Birmingham in 1825, and subsequently in other centres until about 1850 there were some six hundred with a total membership of over one hundred thousand. Their work, however, was severely restricted by the difficulty of finding suitable teachers, for the training of whom no facilities existed, and by the weak educational background of the majority of students. Most of these institutes were eventually replaced by or converted into Technical Schools or Colleges.

Engineering construction about the year 1800 was still carried out mainly in stone-masonry, brick-work, lime-concrete and timber. Iron was used for fastenings and tie-rods, and had, indeed, been used by several Parisian architects in the form of an "armature" of articulated wrought-iron rods embedded in masonry, where tensile forces had to be resisted : and in the pin-jointed frame-work of several boldly experimental roofs. It was, however, expensive, and there was no adequate theory as to how such structures resisted their loads. A more immediately fruitful development was the use of cast-iron as the material for columns and beams. In a paper I read before the Institution in 1949 I quoted Sir William Fairbairn to the effect that the first building with cast-iron columns and beams was probably built in 1801 in Manchester[3]. Since that paper was printed Professor Turpin Bannister has shown that there were at least three such buildings already in existence at that date.[4]

Beams could be and often were proof-loaded before leaving the foundry ; but the testing of columns was beyond the capacity of the appliances available. Some failures occurred with the loss of life and much damage to property. Fairbairn made tests in his own works and later provided facilities for a thorough study of the strength of cast-iron beams and columns by Eaton Hodgkinson (1789-1861) who wrote a classic paper on the subject,[5] and in 1847 obtained the professorship of mechanical engineering at University College, London.

Technical Education in Britain

With few exceptions practising engineers in Britain then, and indeed until the closing decade of the century, paid little attention to theory. They were content with such text-books as those provided by Thomas Tredgold—a "self-educated carpenter and architect's clerk," who in due course became a civil engineer of note. Tredgold gave data obtained by somewhat crude tests. He had also studied the works of Dr. Thomas Young (1773-1829, of Modulus fame) who had embodied in his *Lectures on Natural Philosophy*[6] most of the essential theory developed by French engineers and savants in the previous century with important additions of his own. Tredgold, however, had no facility in mathematics, and at the time his works were accepted in Britain as standard, C. L. M. Navier (1785-1836) had carried the literature in French of engineering theory a considerable step further than Tredgold's authorities. The Universities were slow to admit the study even of science, and still slower to accept that of technology as a suitable preparation for a degree. London only gave its first B.Sc. in 1860, and B.Sc.(Engineering) in 1903. The "dead hand" of Plato, even after two millennia, had a firm grip on the academic world, and intellectual snobbery still looked askance at any intrusion by professors of the "base mechanic arts."

University College, London, opened in 1828. A Professor of Engineering and the Application of Mechanical Philosophy to the Arts had indeed been appointed ; but resigned before the opening day. Some classes in Civil Engineering were held ; but opposition by other departments delayed the actual foundation of the Chadwick Chair of Civil Engineering until 1841. King's College, London, was founded in 1829 ; its department of Engineering opened in 1838. Lewis Gordon was appointed Regius Professor of Civil Engineering and Mechanics in the University of Glasgow in 1840, and was

succeeded in 1855 by J. McQuorn Rankine (1820-1872). It was Rankine who brought out a series of text-books on Engineering, up to the Continental standard, which held their place for half-a-century. A college training in engineering was no great recommendation in the estimation of the mid-nineteenth-century employer. Nevertheless the numbers of university-trained engineers grew, which in due course made it easier to find men qualified to teach mechanical science. The need for such was brought home by the Great Exhibition held in Hyde Park in 1851.

Germany entered the field of large-scale manufacture and export much later than Britain. Her progress was unhampered by old plant and established tradition. German industrialists also made full use of the many keen young men who had received excellent training in the greater Polytechnic Institutes, although, until they were appointed these men had had no actual works or office experience.

By contrast : a British youth wishing to become a civil engineer could, if his parents were able and willing to pay a premium of from £300 to £500, and without any test of his facility in elementary mathematics, science or even arithmetic, enter an office where he would be left to pick up what information and skill he needed as best he could. If he was keen and intelligent, made himself useful and studied in his spare time, he might at the end of his articles be taken into regular employment at a small salary ; but it was nobody's business in particular to teach him anything, and means of instruction were neither easily come by nor reliable. His survival depended on his own efforts.

British engineers laughed off the evidence shown by the Exhibition of 1851, and even more impressively in 1855, that foreigners were overhauling them ; but the Exhibition of 1867 in Paris "shook" at least one observer—John Scott Russell—sent by the British Government to serve as a juryman in adjudging the awards.[7] Hitherto British firms had claimed unquestioned superiority of workmanship and finish for their products. In 1867, not only was the best Continental workmanship as good as British, but in design, economy of material and efficiency in operation, French engines, machines and structures were notably superior. Throughout heavy industry there was no field in which British supremacy was not successfully challenged by one or another Continental competitor.

The British Government had gone so far as to establish in 1853 a Department of Science and Art under the Board of Trade. In 1856 the Department was transferred to the Board of Education. It held examinations in single subjects, and paid teaching centres according to the number of successful candidates they entered. In 1873 the Society of Arts organised examinations in technological subjects, and in 1879 transferred this activity to the newly-formed City and Guilds of London Institute, which has continued from then to the present time to extend the range of subjects.

There were in 1870 ten institutions of University College standing (four in London, two in Scotland, three in Ireland, and one in Manchester) which offered courses in Civil and Mechanical Engineering. Some of them offered Certificates of Proficiency, and one, the University of Edinburgh, was trying to remove the legal difficulty which had so far prevented them from granting a Degree on the results of an examination held at the end of a two-year course. Elsewhere, the provision of classes of instruction had been left to mechanics' institutes, secondary schools and private ventures or benefactions.

Teaching facilities in London were further extended by the action of the City and Guilds of London Institute, which established courses in 1879. These in 1883 were transferred to its new Technical College in Finsbury. Their Central Institution in South Kensington was opened in 1884. The Regent Street Polytechnic was founded in 1882, and others followed in the early nineties. The transfer to technical education by the Charity Commissioners under an Act of 1883 of funds held by London Boroughs for purposes no longer important greatly assisted this expansion of facilities. For the country generally the Technical Instruction Act of 1889 enabled County Authorities to levy a rate not exceeding one penny in the pound to supply or aid technical instruction. In 1890 the "Whisky Money," a fund originally collected by taxes on liquor to compensate holders of revoked licences, was diverted to technical education. In 1904 the Brixton School of Building, the first of the London Monotechnic Institutes, was established.

This extension of facilities for the study of engineering science was certainly not before time. It had become increasingly difficult for the young engineer to pick up the knowledge he required merely by serving as a pupil in an engineer's office and reading Rankine in his spare time. It is true that hundreds of bridges and viaducts constructed during the railway boom of the eighteen-forties and -fifties had been designed only on empirical rules, and that the Royal Commission of 1849 on "The Application of Iron to Railway Structures" had advocated the limitation of its use to girder bridges and arches. Open-frame girders were not widely favoured in Britain although American and Continental designers made free use of them, particularly after the technique of graphic statics had been developed by Carl Culmann (1821-1881), of Carlsruhe Polytechnic, in the eighteen-sixties and later extended by others. Great strides had also been made by French, Italian and German engineers in the application of elastic deformation methods to structural problems. It was time the British engineer tried to catch up.

Many of the modern methods were first introduced to British students by T. Claxton Fidler's standard work on Bridges published in 1887.[8] In 1897 the Institution of Civil Engineers introduced their own examinations for entry into the classes of Student and of Associate-Member.

The Origin of the Institution

Mention has been made of Smeaton's discovery that the raw material of a hydraulic lime always contained some clay. If it did not, there was no reason why clay should not be mixed with the limestone (or chalk) before burning. This mixture was the substance of Joseph Aspdin's patent No. 5022 of 1824 for what he named "Portland Cement," but which, made according to his specification, would now be described as an artificial hydraulic lime. To make what is now known as portland cement the mixed materials must be heated until they form a clinker, and then ground very fine. This was known and acted upon before 1850. The new cement made it possible to produce a much harder and stronger concrete than before ; one which was, moreover, comparatively waterproof, and thus lent itself to use first as protective casing for steelwork ; and then, in combination with steel bars, as the basis of reinforced concrete. Though portland cement was a British invention, and some of the early experiments in reinforced concrete floors were made in Britain, it was in France and Germany that the theory and practice of using reinforced concrete as a structural material were developed : and it was L. G. Mouchel, partner of François Hennebique, who brought this technique to Britain, when he opened an office in London in 1897. The fully

developed steel-framed building was brought to London from the U.S.A. in 1904 by S. Bylander. As these new techniques revolutionised building, and were but poorly catered for by the older institutions, the Concrete Institute was founded in 1908. From the start many of its leading members felt that it was sound engineering not to bind oneself to the use in all circumstances of one material when several were available. The Structural Engineer should understand concrete, steelwork, masonry and timber. The first World War held up an earlier action ; but in 1922 the Concrete Institute widened its range and re-made itself as the Institution of Structural Engineers.

It is interesting to recall that the first course in Britain of instruction in reinforced concrete, including practical work, was started at the Brixton School of Building in the session 1911-12 under H. Kempton Dyson, the Secretary of the Concrete Institute. In the following session a course in the Theory was started at Westminster Technical Institute under Ewart S. Andrews. In 1913-14 I attended his lectures as a student.

Under the new Articles the classes of membership of the Institution were laid down very nearly as they still remain. Qualifying examinations were mooted in 1915, but did not materialise until May, 1920. As set in 1922 the examination syllabus remained practically unchanged until 1944. Thereafter, in the Graduateship Examination five short papers in sciences were replaced by one longer paper in Structural Engineering Science, and in the Design paper in the Associate-Membership Examination a general question was added to the alternative questions in Steelwork or Reinforced Concrete. The Education Committee have for several years felt that the time has come to modify the papers in these two subjects again. At this point in my address I should have liked to outline the modifications proposed ; but, as the matter is still under discussion in Committee, such action would be premature. While speaking of examinations, however, it might not be inappropriate to consider just what a candidate's success or failure in an examination is intended to convey to the body which requires him to sit for it.

Examinations

The most reliable form of examination is one in which a candidate is called upon to show whether or not he can perform with the necessary skill certain operations which will be part of his job. In such a test actual marks are unimportant ; but, if given, the standard for a pass must be high. But many examinations do not directly test any applicable skill, but rather the progress made in the attainment of basic knowledge. There may be wide differences between the courses of study the various candidates have pursued, and in the duties they will carry out after qualification. The examiner must set alternative questions and, although these may be allotted equal possible marks, they cannot all be of equal difficulty.

That an examination of essay type in particular is extremely difficult to mark with strict fairness to every candidate was strikingly shown in a very careful investigation made by the British Committee of an International Conference on Examinations, which met in Eastbourne in 1931.[9] This Committee persuaded several examining bodies to allow actual scripts, with the marks obliterated, to be re-examined by panels of examiners who did not see each other's marking. When the results were compared they showed startling variations both in the level of the marks awarded to each candidate, and in the order of merit in which each examiner's marks arranged the candidates. It was clear that those who might either pass or fail, according to which examiner marked their scripts, formed a very broad band. Moreover, the same scripts, re-examined after an interval of time by the same examiners, showed further surprising variations.

Greater consistency would be expected in the marks of an examination in which the questions called for factual answers backed by the application of mathematical skill than in those given for essays and the like. Probably the fairest written examination would be one in which the questions were easy but the marking strict and the pass marks high. But examination papers are usually published, and are liable to be compared with those of other examining bodies, and considered for purposes of exemption. As a matter of prestige examining bodies and their moderators are prone to encourage examiners (who are nothing loth to comply) to set impressively difficult questions.

When the examiner who has set a stiff paper sees the answers it has evoked, he is liable to be faced with two unsatisfactory alternatives : either he must fail all but a few particularly able candidates, or he must mark so leniently that the marks he gives to the average candidate will add up to not less than a low-standard pass-mark, even if that pass-mark is attained by answers so incomplete, vague or erroneous as to be vocationally worthless. Of course no candidate passes a qualifying examination on one paper only, but in a group of subjects. Also, in most important examinations, such as those of the Institution, all borderline cases in every subject are reviewed by a second examiner. Even so, there are critics who contend that a better test than the written examination paper could and should be devised. Some, for instance, would favour the interview.

The Committee, who studied the marks of examiners, included in their investigations an interview of sixteen candidates by two independent boards of experienced interviewers. Both boards had before them the candidates' records. Both appeared to secure the confidence of the candidates, who spoke with freedom and frankness. The questions brought out the weaknesses of candidates ; but the reporters considered it was largely a matter of chance whether the interviewers struck on a topic on which the candidate felt so strongly that he was able to display his individuality. The candidate who was placed First by Board I was placed 13th by Board II ; the candidate placed First by Board II was placed 11th by Board I. The substantial prize offered to the most successful candidate was awarded to one placed Second by one Board and Fourth by the other.

An interview by a single examiner may be more intimate than one conducted by a board ; but it leaves the candidate's fate in the hands of one man. Every experienced interviewer tends to consider his own judgment of men to be infallible, whereas he is in point of fact less influenced by the quality of the candidate in an impersonal sense than by the ease or difficulty with which two individuals of probably different, and possibly antagonistic, types can establish a mutual sympathy. Such an interview is invaluable between a senior and a junior who, if accepted, will work directly under him. An interview is also a useful means of checking information given by a candidate on an application form, or the authenticity of work he has submitted as his own ; but as a means of assessing personal qualities its validity is low.

There are psychological tests which have proved valuable as aids to the selection of officers in the fighting Services. So far, to the best of my belief, they have not been tried by any professional institution. It would seem that, with all their faults, we must continue to set

written examinations, which could be supplemented by the record of a candidate's work as a student : and in this respect an internal examination has the advantage over an external. This was recognised by the Board of Education when in 1911 they discontinued the Science and Art Department Examinations, and encouraged Technical schools and colleges to organise group courses, and to examine their own students. Since 1918, also, the tendency has grown for professional bodies to associate themselves with the Board (now the Ministry) of Education in the approval of group courses, the assessment of examinations, and the endorsement of National Certificates of Proficiency. The Institution is represented on the Joint Committees on Higher National Certificates both in Civil Engineering and Building, and accepts these certificates, duly bearing our endorsement, in the same way as it accepts University Degrees as exempting qualifications from the whole of the Graduateship Examination ; and also in certain cases from the Theory paper in the final Associate-Membership Examination. Successful students in the Internal Examinations at the end of approved three-year full-time courses in certain technical colleges are granted the same exemption. At present this privilege applies to colleges at Brighton, Brixton, Hammersmith and Salford; but it will shortly be extended to others. The Institution is represented on the Advisory Committee on Structural Engineering of the City and Guilds of London Institute, and grants similar exemption to holders of that Institute's First Class Full Technological Certificate in Structural Engineering.

A candidate exempted from part of the Institution's own examination must also produce evidence of a good general education, or submit to the Common Preliminary Examination held by the Engineering Joint Examination Board, of which ours is one of the constituent Associations, and your new President is Chairman.

This last requirement is sometimes onerous to the student who has won his Higher National Certificate by five years of steady application to evening study, but who did not before leaving his day school obtain a General Certificate of Education or its equivalent. It is to be hoped that such cases will be fewer in future, particularly as, from this year onwards, scholars in Technical Schools can be examined for the General Certificate of Education by the newly-constituted Joint Examination Board, which has their requirements particularly in view. As one of the two members of the Board chosen to represent the professional Institutions, I have seen the syllabuses and sample examination papers, and regard this provision as a most important facility. There is no reason why the Technical Secondary School should be in any way inferior in its standards to the Grammar School and, as its choice of subject-matter and method of treatment, even of the same subjects, can be much more suitable for the boy who thinks both with head and hand, there is every reason why good students [illegible] encouraged to pursue a technical [illegible] ollow the traditionally scholastic [illegible]

Towards a Liberal Technical Education

It is perhaps vain to discuss the relative merits of alternative means of schooling unless one is clear as to the ends which that schooling is expected to serve. My own view of the matter is as follows : every new generation is a fresh invasion of barbarians, and, if the extreme exponents of "free discipline" had their way, the invaders would all grow up barbarians, as, indeed, some of them do anyway. The object, then, of primary education is to condition the child to that form of superficially civilised life which his elders prefer. The aim of Secondary and Higher Education is to enable him to earn a good living, and to take his place as a citizen. I make no apology for bracketing these two together as a single aim. No man can be a good citizen or a happy individual unless he can earn the respect of those he admires, including himself : and the surest way to do this is to prove that, with the full and at times strenuous exercise of the skill and energy at his command, he can cope with a job that he thinks is worth keeping. In a highly organised and differentiated community there are many varieties of service to be performed, and, as individual aptitudes vary widely both in nature and quality, it is a matter of immense importance for the public good and for his own satisfaction that the right person should be trained to do the right job—and trained in the most expeditious way.

At the present time, and, indeed for many years to come, industry and the professions, especially that of teaching, will absorb every capable scientist, technologist and technician, who can, with the facilities now available, be trained. There are, also, many high administrative posts which should be filled by men with advanced knowledge of Science, or of some branch of Technology, but which are not so filled because of doubt as to whether such men have the breadth of outlook required of administrators. There is a crying need that men with the very best brains should be selected and trained in science or technology. It is pathetic to see so many of the most promising young scholars being urged by headmasters to pursue lines of study which will, at the end of the university course, prove vocationally "dead ends." Far be it from me to suggest that the study of Arts subjects should be discouraged ; but, whereas it is always possible to staff teaching posts in the Arts subjects with scholars of high quality, since competition for their services elsewhere is seldom unduly keen, there is a grave danger that, because far too few graduate teachers of Science in sixth forms are available, the supply of technologists and scientists may be strangled at its source.

There will always be scholars who find in study and investigation the satisfaction of an insatiable curiosity to know more and more about things which have no bearing on the way in which they make their living. When learning was pursued at their own expense by people with private means, this satisfaction could be urged as the main incentive to study. But, since the State has confiscated private means and maintains the scholars, the satisfaction of personal curiosity has become a hobby for scholars and not their main occupation. Fortunately as a hobby it still has many ardent devotees.

Scientists and technologists, especially those who will rise to the top of their profession (and at the start no one knows which they may be) cannot be adequately trained on concentrated doses of science and technology only. All science and technical courses need humanising. Some teaching institutions have attempted to do this by inserting in the curriculum classes in such subjects as English Literature or Philosophy. It is doubtful whether such insertions will meet the need. There is one line, however, which appears to me to be much more promising : the study of the history of the main subjects of the course, science, engineering, building or the like.[10]

With whatever subject one may start, its history, studied with even a little imagination, immediately raised questions which carry enquiry far beyond the subject-matter of the study from which it stemmed. Almost any student of any subject can have his curiosity aroused by historical questions, the answers to which

explain what he does now in his own job, why he does it in a particular way and how his profession or craft arose out of and contributes to the life of the community to which he belongs. The immediate difficulty would be that the teachers, themselves, do not know their history, although in some University courses steps are already being taken to remedy this defect. As a source book *The Transaction of the Newcomen Society for the Study of the History of Engineering and Technology* (of which twenty-six volumes have been published) are unrivalled. There are also, now, a few useful textbooks.[11] It is not suggested that History should become a subject of examination for engineers ; but its introduction as a subject of study could have far-reaching beneficial consequences, and go far to provide the liberal element in technical courses, the absence of which is so often, and not unjustly, deplored.

Finale

Instruction and study, however well conducted, can never make an engineer : nor can scholastic success, or its lack, be taken as proof that one man will make a good engineer, and another will not. Experience in the shop, the field and the drawing-office and ability to profit by the human contacts they bring, are equally necessary and important. In judging the suitability of an applicant for Corporate Membership of the Institution his practical training and the first steps towards his career are therefore considered equally with his academic success. The Membership Committee always bear in mind that we want to welcome as many new members as possible, but that, in fairness to those already in, and to the public, the Institution's stamp must not be placed on anyone likely to prove unreliable.

We have, nevertheless, built up a membership seven thousand strong, and still increasing. The Institution has established its right to speak with authority on behalf of our profession both in respect to matters of interest to members, and on matters of public interest in which their special knowledge is of importance. It has, as I have tried to show in my somewhat discursive remarks, its roots deep in the past. The future is unknown. The Institution and its members have played no mean part in their country's service in two World Wars ; and should there come a third would do so again—and afterwards—however bleak the prospect. If, however, as men of goodwill everywhere are trying to ensure, that calamity be avoided and a period of peace and prosperity lie before us, we stand, fully equipped and ready to give our best to provide structures which we may leave to posterity to compare not unfavourably with the achievements of our great forerunners.

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Book Reviews

Strength of Materials : A Text-Book for Degree Students, by G. H. Ryder. (London : Cleaver-Hume Press, Ltd., 1953.) $7\frac{3}{4}$ in. × $5\frac{3}{4}$ in., 278 pp., plus x, 21s.

This book, based on the syllabus of the University of London, may be warmly recommended to students taking a Degree course.

As each topic is introduced, numerical examples are given ; a total of some 130 worked examples appear in the text, and there is an adequate supply of problems, with answers, at the end of each chapter. The treatment is everywhere lucid and is generally well illustrated by diagrams.

In producing a text-book well suited for self-instruction, the author perhaps gives the student the impression that all problems of applied elasticity are now completely solved, although this may be unavoidable in a volume of this nature. The publishers are to be congratulated on producing such a handsome book at moderate cost.

E. M.

La Methode de Cross, et le Calcul Pratique des Constructions Hyperstatiques. (The Cross Method, a practical method of calculation for hyperstatic constructions), by P. Charon. (Paris, Editions Eyrolles, 1953.), pp. 300, 6 in. × 10 in. 3,800 francs.

The Cross method brings considerable changes in the study of hyperstatic constructions and the usually long and involved calculations may be replaced by simple and rapid computations leading to rigorously correct answers.

This book shows how the Cross moment distribution method is derived from the Equations of Bresse. It is designed for immediate applications on all hyperstatic structures, from the simple portal to the skyscraper. It can be applied to straight and curved beams with constant or variable sections subjected to fixed or moving loads.

The main chapter is devoted to redundant frames containing straight beams of constant sections, and shows how the method works in its application to frames with or without lateral restraint, to non-rectangular frames, and to Vierendeel girders. An application of the method to temperature stressing is also given.

Several alternative moment distribution methods, derived from the original, are explained in the following chapters.

The Cross method has made it possible to analyse, in the elastic range, some very complex types of structures which are not easily amenable to treatment by the classical formulæ. The method is consequently gaining much ground.

The book is eminently practical and contains numerous examples of application.

P. G.

Fabrication and Erection of Steel Plate Girder Railway Bridges*

By M. F. Palmer, M.I.C.E., M.I.Struct.E.

Introduction

The railway bridges mentioned in this paper were renewals of existing bridges, and as such, it was essential to erect them during possessions of the tracks measured in hours. The methods of erection varied according to site conditions.

The paper describes the fabrication of two rather unusual types of all-welded plate girder bridges, in which completely welded spans were despatched from the fabricating works and lifted into position on site by cranes.

An important feature of the first bridge, which has eleven units bolted together side by side with close-fitting bolts, is that the reamering out of holes after shop assembly was avoided.

The second bridge is believed to be the first all-welded bridge in this country to be curved to conform to the curvature of the tracks.

[illegible] rolling-in method, of a riveted [illegible] example of erection by jacking-up a [illegible], are described.

Stewarts Road Bridge

The general arrangement is shown in Fig. 1, from which it will be seen that the brid[illegible] constructed of ten single track spans and a walkway span. Each of these spans was completely welded in the works, delivered to the site and placed in position by means of two 36-ton locomotive cranes standing on an adjacent track.

An outstanding point is that each span is in the nature of a trough as shown in the enlarged cross-section given in Fig. 2, and the units are bolted together with $\frac{7}{8}$-in. diameter turned bolts in 29/32-in. diameter holes.

Owing to the narrow space of about $16\frac{1}{2}$ in. between the web plates of adjacent units, it was desired by the contractors to eliminate any reamering of holes in the stiffeners after the units were fabricated. To achieve this, all the stiffeners and joint covers were jig drilled. For the purpose of the assembly of each complete main girder, comprising two half girders, temporary cover plates were made which were wider than the actual covers used in the completed bridge. The purpose of these wider cover plates was to increase the distance between the web plates from $16\frac{1}{2}$ in. to 26 in. whilst each complete girder was being welded in the jig. Fig. 3 is a photograph of the actual jig in use. In this picture, the upper web plate has not yet been placed in position and the temporary cover plates can be clearly seen. The holes in these temporary covers were made 57/64 in. diameter, i.e., 1/64 in. less in diameter than the holes in the permanent covers so as to allow, in effect, more clearance between the bolts and holes in the permanent work.

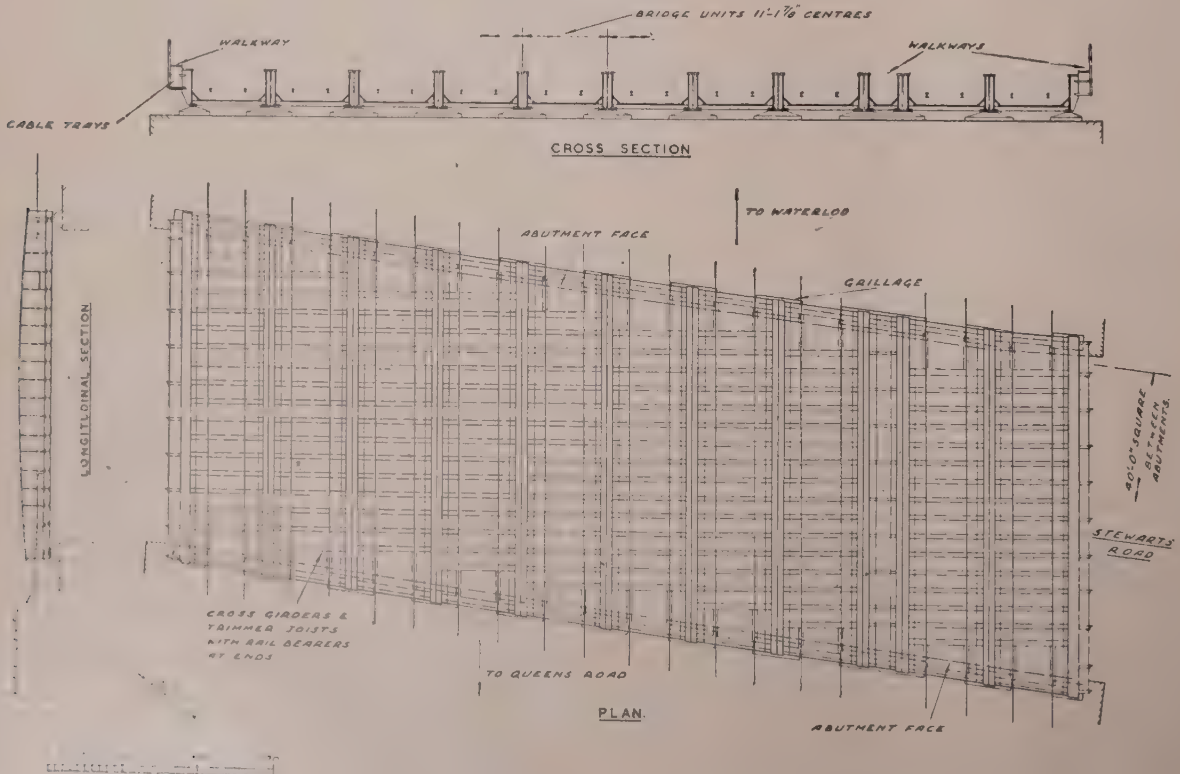

Fig. 1.—Stewarts Road Bridge. General arrangement of spans

Paper to be read before the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1, on Thursday, December 16th, 1954, at 6 p.m.

As much welding as possible was done in the jig in the position shown, after which the whole girder was removed while still bolted together and turned over for completion of the welding. The two outer welds on the bottom flanges of these girders were not done in the jigs ; it was decided to leave these until the cross-girders were in position to assist in reducing distortion. The two halves of each girder were then taken apart and the permanent joint covers bolted in position, making the complete girder its proper width.

Both top and bottom flanges were from 18-in.-wide Universal plates split down the centre by machine

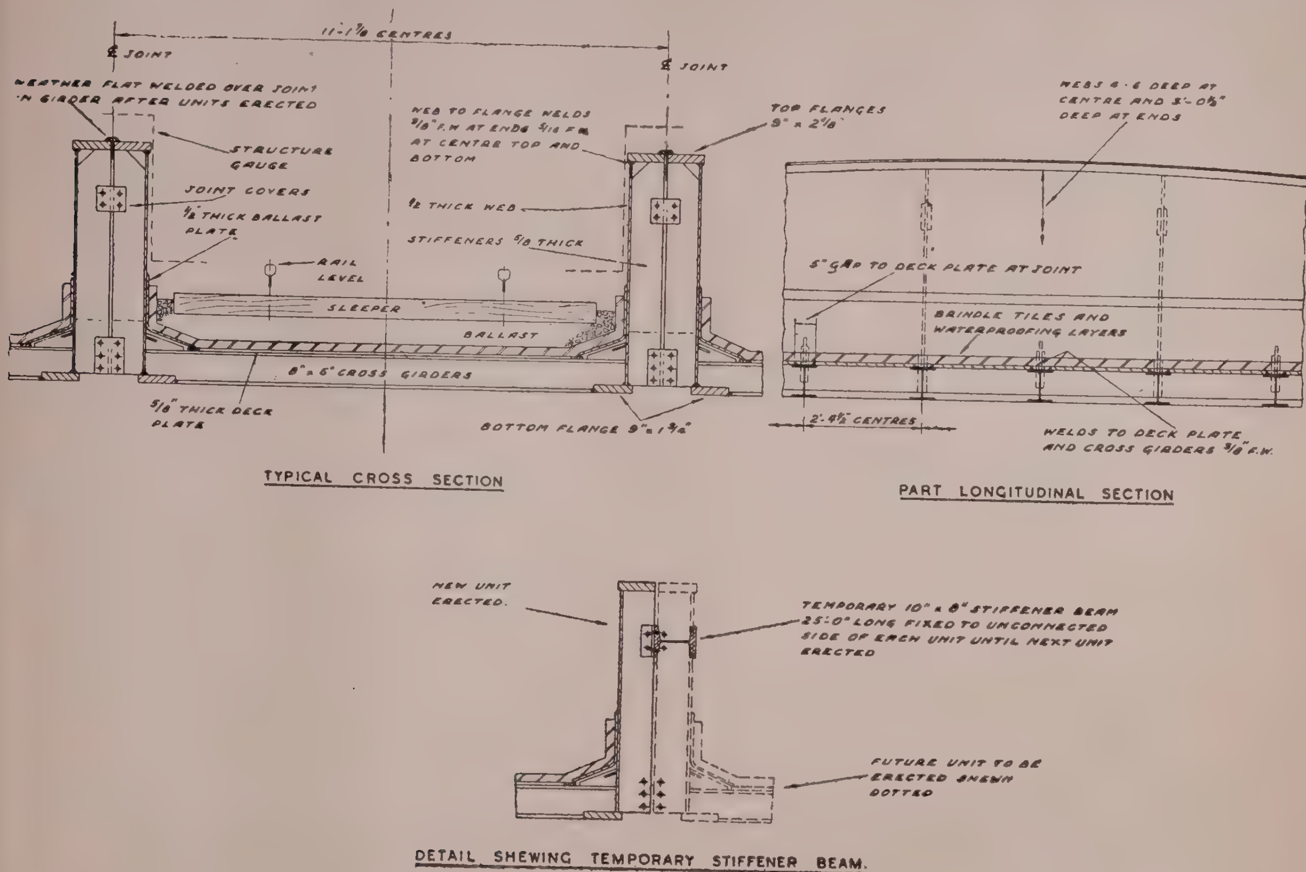

Fig. 2.—Stewarts Road Bridge. Constructional details

Fig. 3.—Stewarts Road Bridge. Assembly jig for girders

Fig. 4.—Stewarts Road Bridge. Welding of deck

burning. They were subsequently straightened and the top flanges curved as required. In view of the flange thicknesses, low hydrogen electrodes were used. Some difficulty was experienced by the welders at first in making satisfactory fillet welds with these rods in the horizontal-vertical position, but this was overcome after they had received special instructions from the electrode manufacturers. All flanges were left long to allow for

shrinkage and were cut to the correct length after the welding was completed.

The two outer parapet girders had parallel flanges, and as will be seen from Fig. 1, the flange to web welds were near the edges of both top and bottom flanges. The contraction of these welds produced curvatures which

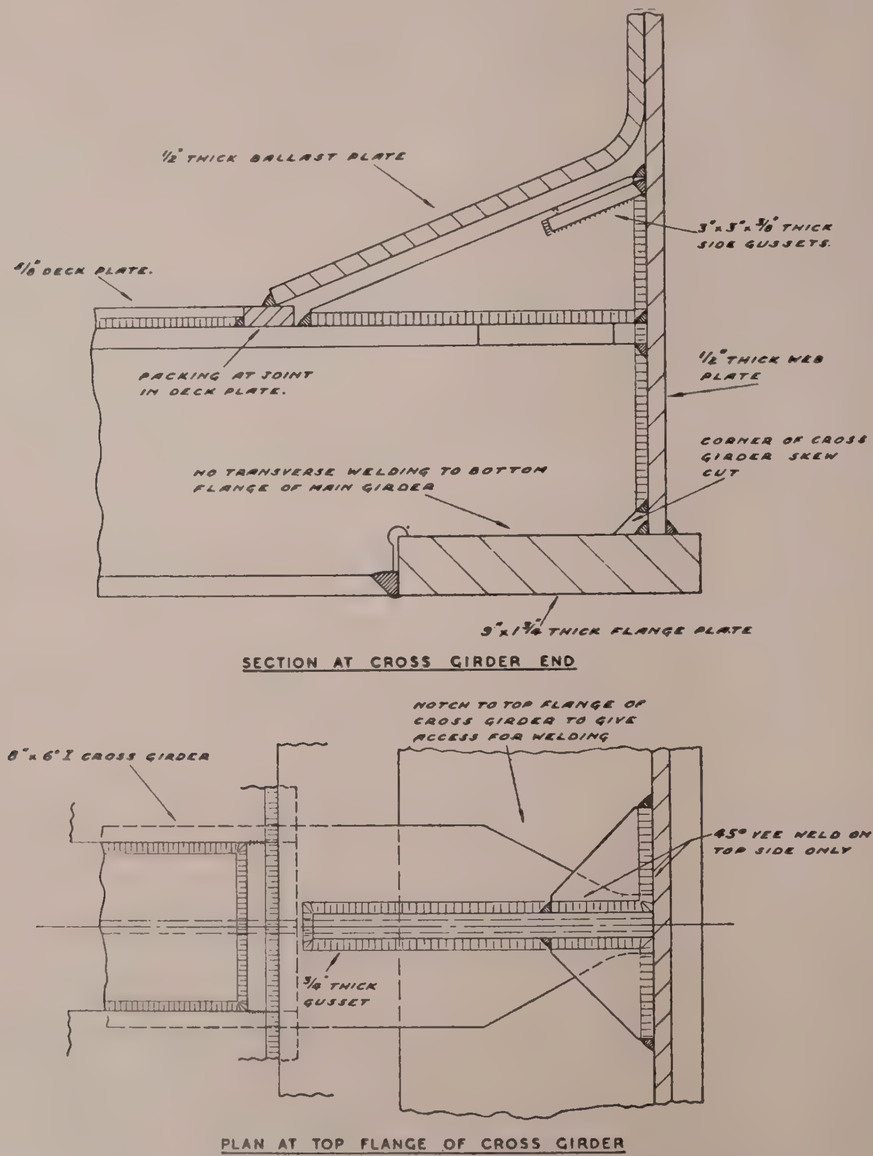

Fig. 5.—Stewarts Road Bridge. Detail at end of cross girders

were corrected by passing the girders through a straight- [illegible].

[illegible]he first span was assembled on stallages and the [illegible]oor system welded. By agreement with the Engineer, certain alterations were made from the original drawings, the chief one being that the floor plates which were [illegible]d together at the joints were [illegible] in. cross-girder and fillet [illegible] [illegible]he overhead welding on the [illegible] deck was eliminated.

[illegible]he welding of th[illegible] shown in progress in Fig. 4. During welding, the distan[illegible] between the top flanges of the main girders was incr[illegible]sed by about ½ in. over the required dimension to all[illegible] for the contraction effect of the welding of the deck. Two of the horizontal struts used for this purpose can be seen in the photograph.

A detail of the end connections of the cross girders is given in Fig. 5. Precautions were taken to avoid welding across the bottom flanges of the main girders except at the supports. To reduce any concentration of stresses at the junction of the top of the triangular gussets with the web plates, the sloping 3 in. × ⅜ in.-thick side gussets shown were introduced.

Having completed the first span, this was used as a base for the building of the second span. The adjacent half girders were completely bolted together with turned bolts, and the joint between these girders was not broken until the second span was completely welded. This system was repeated for all subsequent spans, and when the units were eventually brought together at site, no difficulty was experienced in replacing the turned bolts in the holes. Using this experience as a guide, it should be possible to fabricate quite large structures in the shops, where certain members to be built into the structure have been joined together with turned bolts before any welding is done. After complete fabrication,

they could be broken down for transport and re-assembled on site using the same bolts.

The sequence of operations in the construction of the spans is clearly shown in Fig. 6. In the foreground is a completed span, painted and ready for despatch. A second completed span is behind it, on which is being built a third span. The clean lines of the completed spans are striking and should facilitate future maintenance and painting. The underside of the bridge

Fig. 6.—Stewarts Road Bridge. Assembly of spans in shops

oresents a very clean appearance and the insides of the irders are readily accessible for painting.

The intermediate girders were hog-backed to reduce bstructions to the permanent way as much as possible

It should be pointed out that owing to the thinness f the bearing plates, some trouble was caused at first y distortion due to welding, and these were subseuently increased in thickness. Bearing plates should e of adequate thickness to avoid distortion of the achined surface. A thickness of at least 1½ in. would e advisable, and the size of welds kept to a minimum.

One condition of the contract was that in view of the riginality of the design, tests should be carried out in he works to determine the stresses in the members and elds.

In the first test, a load of 65 tons, made up of steel abs supported on timber sleepers and placed so as to revent arching, was concentrated on the middle third one span.

The second test was a repeat of this, as a check on the adings.

For the third test, the full length of the span was aded with 120 tons made up in a similar manner, and r the fourth test, the same load was concentrated on e middle two-thirds of the length.

After the above tests were completed, it was decided carry out a further test of the cross-girder connections th two adjacent tracks loaded. The arrangements ade for this test are shown in Fig. 7, from which it ll be seen that in this case the loading was applied by eans of hydraulic jacks.

From the very large number of electrical strain gauge adings taken, it was ascertained that under full loading ere would be no excessive local stresses.

Each unit was subsequently delivered to Nine Elms Goods Yard by road and transported as an out-of-gauge load by rail to the site, a distance of about half-a-mile.

An erection programme was drawn up before site work commenced and was strictly adhered to. This was necessary since the use of railway cranes was required on practically every week-end over a period of four months or so. One new span was erected every fortnight, the intervening week-ends being used for taking out the existing spans and inserting temporary waybeams for one or two spans ahead.

After each new span had been placed in position at site, a temporary 10 in. × 8 in.-joist stiffening beam 25 ft. long was bolted to the stiffeners on the outer half girder. This had the effect of doubling the width of the compression flange until the next span was connected to it. The arrangement is shown in Fig. 2.

It was anticipated that with this number of units side by side, some growth would take place, and to cover for this, the walkway span was reduced in width by a matter of 1 in., but it was found upon completion that this provision need not have been made.

The space of ½ in. between the two girders was covered by a 2 in. wide strip welded on at the site. Apart from this and the welding of some brackets to the parapet girders, there was no other site welding.

While the spans were at Nine Elms awaiting erection, the decks were waterproofed and tiled and a small amount of concreting at the ends completed. The existing abutments were used.

The overall sizes of the main units for transport were 47 ft. 6 in. long and 11 ft. 2 in. wide, the maximum lift being 25 tons.

Twickenham Fly-over Bridge

This bridge replaces the original wrought-iron structure, which was about 70 years old. Whereas the old bridge was constructed of three straight spans forming chords to the curvature of the track, the new bridge is curved to suit the track. Owing to the steep gradients of the approaches to the bridge, the new structure had

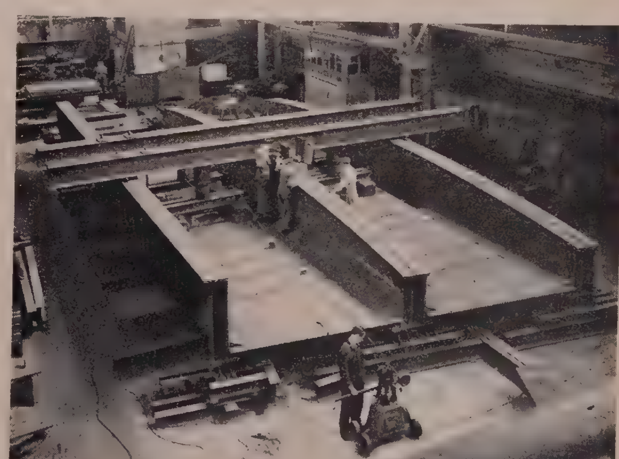

Fig. 7.—Stewarts Road Bridge. Testing cross-girder connections

to be designed with the least possible depth of construction. This was achieved by curving the main girders and thus reducing the span of the cross-girders to a minimum. The curving of the main girders did not lead to an increase of the total amount of steel required, although the outside girders are stronger than the inside ones.

The general arrangement of the bridge is shown in Fig. 8. The existing abutments were re-used with

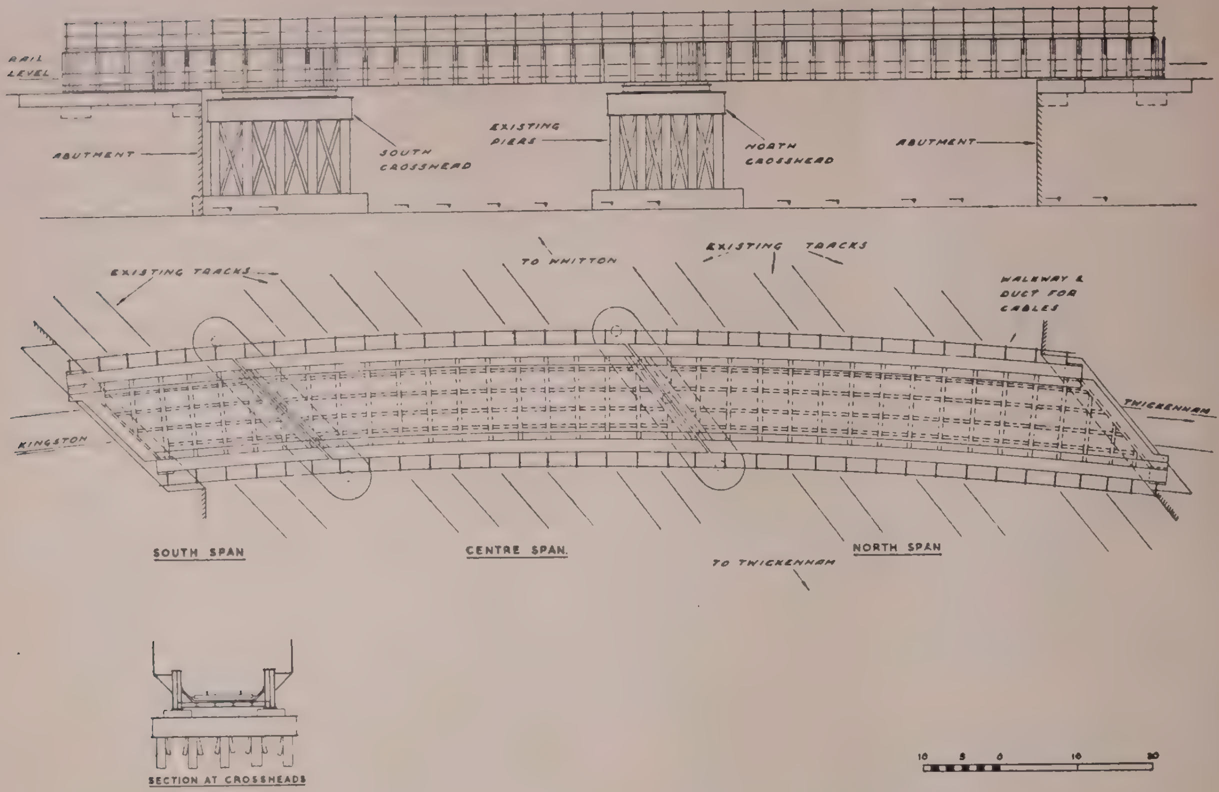

Fig. 8.—Twickenham Fly-over. Arrangement of new bridge

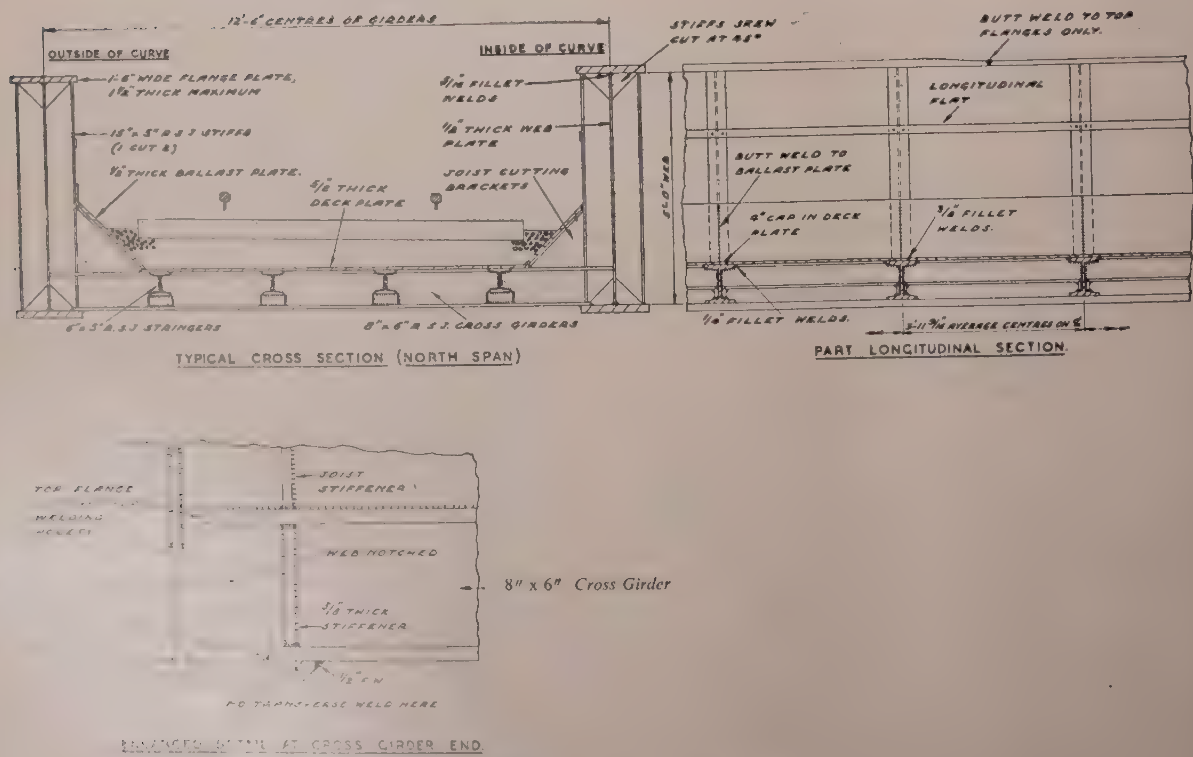

Fig. 9.—Twickenham Fly-over. Constructional details

modified precast concrete caps ; the two intermediate iron trestles also remain, but are fitted with new welded crosshead girders.

Fig. 9 gives a typical cross-section of a span and an enlarged detail of the cross-girder connections. The tension flanges of the main girders were 18 in. wide and varied in thickness from $\frac{3}{4}$ in. to $1\frac{1}{2}$ in. It was desired, as a precaution against brittle fracture, to avoid any transverse welding on these flanges, except at the supports. For this reason, there are no transverse welds at the cross-girder connections, and the tension flanges, although 62 ft. long in the maximum case, were obtained in one length from the mill to avoid a joint.

The question of brittle fracture assumed greater importance soon after this contract was placed, and the contractors were asked by the Engineer to obtain the tension flanges to the following specification.

The carbon content to be limited to 0.2 per cent., carbon manganese ratio to be not less than 1 to 3, and test pieces cut from the flange plates to pass an impact test of not less than 15 ft. lb. at —20°C.

The contractors found that the mills were not able to accept these conditions in their entirety. They were willing to roll the steel to the carbon requirements but could not undertake the impact test. To overcome this, the contractors had to accept the plates in this instance and bear the risk of rejection of the material should the impact tests fail. These impact tests were carried out by the British Transport Commission's own staff, and fortunately were found to be satisfactory.

Fig. 10.—Twickenham Fly-over. Girder assembly jig

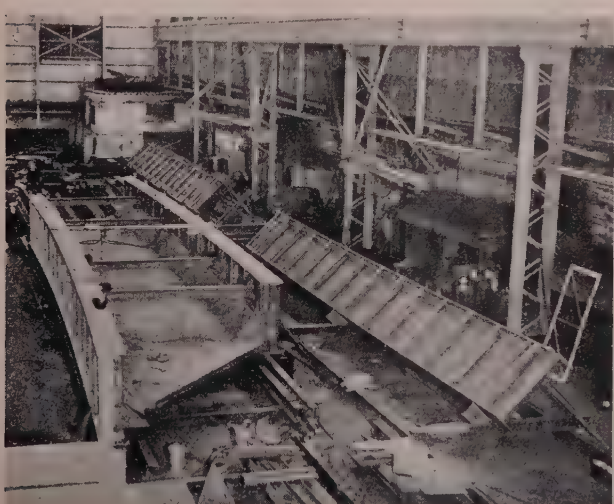

Fig. 11.—Twickenham Fly-over. Assembly of one span in shops and girders for further span in jig and cradle

The plate girders for this bridge were fabricated in the jig shown in Fig. 10. This jig had a series of adjustable supports for the web plates which enabled the webs to assume their correct curvature when placed in the jig.

Fig. 12.—Twickenham Fly-over. Completed span *en route* to site

The flanges were curved by machine before placing in position.

The top flanges were in three lengths, which were welded together before placing in the jig, and the same procedure was adopted for the joints in the web plates.

Special provision was also made in the jig for supporting the stiffeners on the underside of the girder. Whilst the girder was in this position, all the component parts were tack welded, and on the upper side the bottom web to flange weld was completed, together with the continuous welds to the upper stiffeners. Low hydrogen girders and longitudinal trimmer joists were completely welded at their intersections before the plating was placed and welded. Low hydrogen electrodes were used for most of the deck construction, and contact electrodes for the overhead welding on underside of the deck plates.

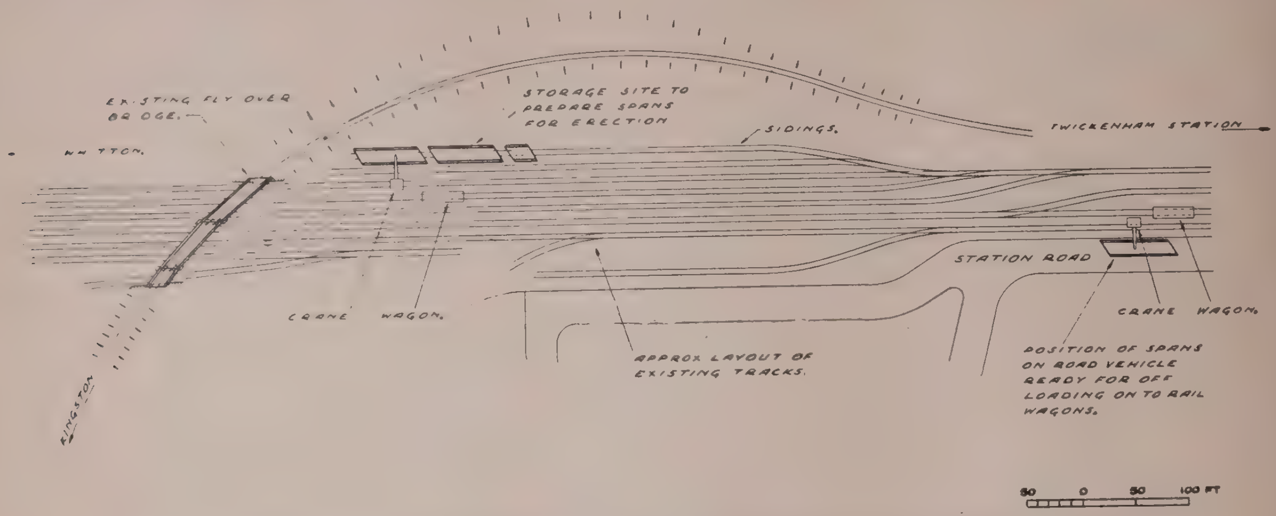

Fig. 13.—Twickenham Fly-over. Site plan showing existing bridge, off loading site for new spans and storage site

electrodes were used for the web to flange welds and contact electrodes for the stiffeners.

The girder was then removed from the jig, turned over by cranes, and placed in an adjacent cradle, consisting simply of three supports. In this position, further welding was completed. Two further turns were necessary to complete the girder. The webs and flanges were left long to allow for contraction due to welding, and

Fig. 14.—Twickenham Fly-over. One of the spans being lifted by crane

[illegible] at a suitable opportunity. [illegible] and one in the cradle.

[illegible] of the girders, the [illegible] the versine of the [illegible] of about $1\frac{1}{2}$ in. in [illegible] curving the flanges [illegible] of the girders without [illegible] the complete span.

The spans wer[illegible] 3 ft. high to enable the [illegible]eck to be carried out in fair comfort. The main girders, cross-

Fig. 11 also indicates the first span nearing completion and shows the transverse struts which were used to keep the top flanges of the main girders slightly open to allow for contraction due to the welding of the deck.

Since this bridge would on completion be placed over tracks carrying steam traffic, it was deemed necessary to seal completely all the joints on the underside of the deck with welding in order to reduce the effect of corrosion to a minimum.

On the completion of the welding of the spans, it was found that the maximum variation in the versine of the curvature of the flanges from the theoretical figure was $\frac{1}{4}$ in.

On this bridge, all bearing plates were bolted to the main girders.

The "square" spans c/c bearings, and weights are given below.

	Square Span	Weight of Steelwork only
North	60 feet	34 tons
Centre	52 feet	30 tons
South	23 feet	13 tons

Owing to the skew ends and curvature, the maximum size for transport was 75 ft. long, 15 ft. wide and 5 ft. 6 in. high.

In Fig. 12, one of the spans is shown *en route* to site. The use of two independently steered trailers enabled sharp corners to be negotiated without difficulty. The two transverse struts are part of the lifting gear.

The bridge was originally designed for despatching in halves, having a joint on the longitudinal centre line, the arrangement being that both halves of each span would be brought together on a stallage adjacent to the site and welded prior to lifting in position. After making detailed investigations, however, the contractors found that it was possible to send each span away completely welded. The possibility of transporting these spans on rail was the governing feature. The layout of the tracks and roads adjacent to the site of the bridge is shown in

Fig. 13. Station Road runs parallel to the tracks at one point, and by placing a locomotive crane on the nearest track, each span could be lifted off the road vehicle and landed on a rail wagon standing on the adjacent track as shown. The wagon containing the span was shunted over to the far tracks, the span was then lifted off by crane and landed in its temporary storage position. Fig. 14 shows one of the spans being placed in this position.

Whilst on the stallage adjacent to the site, the decks of the spans were waterproofed and tiled. The final painting of the units was also carried out at this stage.

The work of dismantling the existing bridge, the erection of the new bridge, ballasting of tracks, etc., is intended to be done by the British Transport Commission's own staff during a total possession of the tracks for 32 hours. The lifts will be by locomotive cranes of up to 45 tons capacity standing on the tracks below.

Erection of Bridges

The erection of two bridges by lifting in complete spans, has been described, and details of erection by other methods of two further bridges follow. The constructional steelwork details of these latter bridges are on more orthodox lines and need not be specially described.

New Cross Road Bridge

This bridge spans the Grand Surrey Canal, which is about 70 ft. wide with a towpath on one side. There are two short approach spans on each side giving access to local properties. The old bridge built in 1869, was of lattice construction and showed severe deterioration in the bottom booms of the main girders which were in the form of a channel, and although drainage holes existed these members held water at various points. The double tracks of the bridge were carried by longitudinal timbers set in troughs in the iron-plated deck. These troughs also suffered from severe corrosion.

The approach spans which comprised longitudinal wrought iron and cast-iron girders, were supported on temporary timber trestles whilst the caps of the piers and the tops of the abutments were renewed. They were subsequently replaced in two possessions by precast units formed from steel beams encased in concrete and lifted in by crane.

The new steel main span consisted of three riveted hog-backed plate girders, with 10 in. × 8 in. × 55 lb. joist cross girders encased in concrete except for the underside of the bottom flange. Permanent precast concrete slabs 2 in. thick resting on the bottom flanges of the joists formed the shuttering.

The rolling-in method of erection was adopted, and Fig. 15 shows a general layout of the works at the site. Trestles to support the rolling-in paths were constructed, one on the towpath and the other on piles driven into the bed of the canal. These trestles were extended on either side of the existing bridge sufficiently to enable the new span to be constructed on one side prior to rolling in, and the old bridge to be rolled out on the other side during the change-over.

A view of the trestle on the towpath during construction is given in Fig. 16, and that of the piled trestle in Fig. 17. To avoid driving piles through the deck of the existing bridge during short possessions of the tracks at night, two plate girders were used to span under the

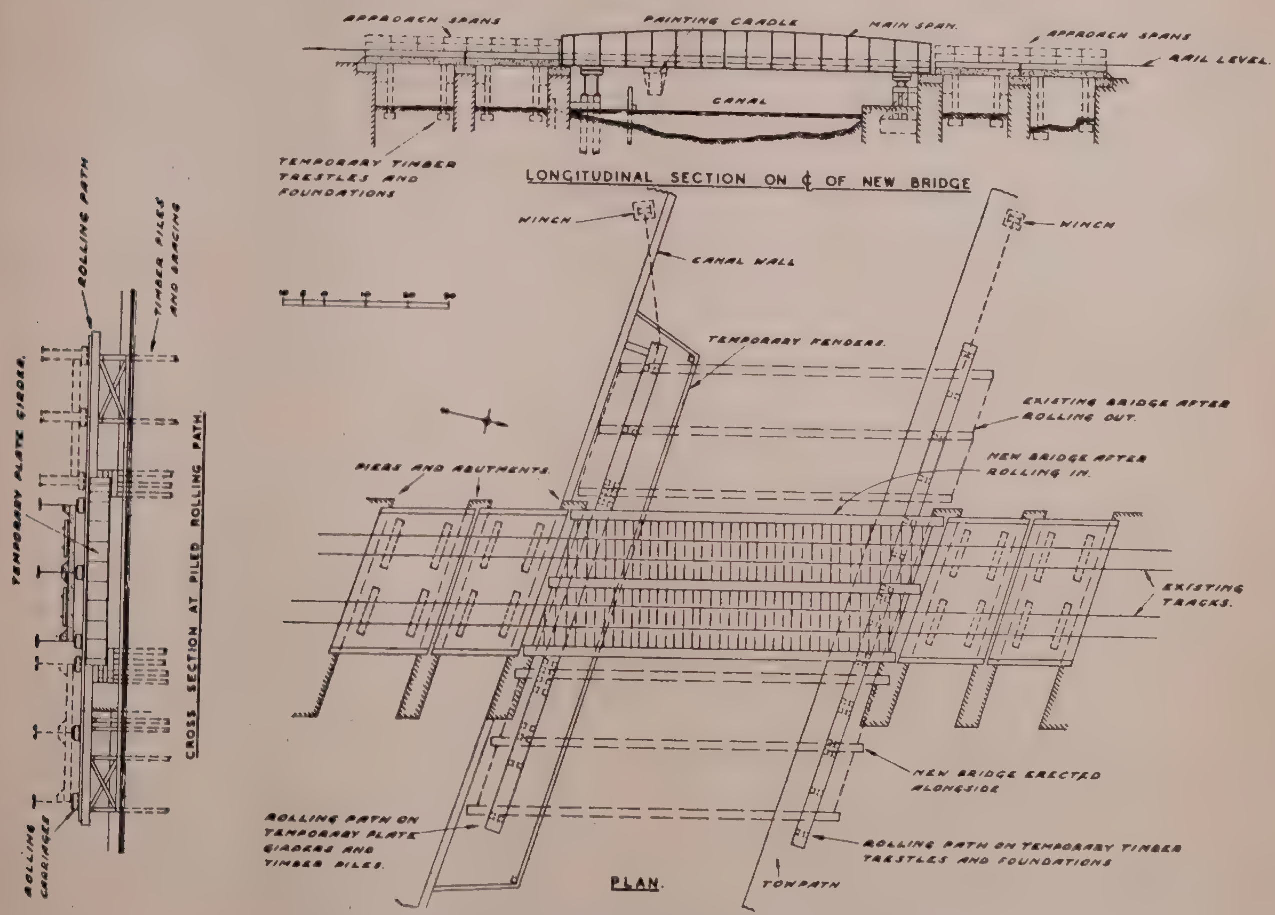

Fig. 15.—New Cross Bridge. General layout of site works

existing bridge and were supported on two groups of piles driven clear of the bridge.

The three main girders for the new bridge, each about 93 ft. long, 9 ft. 6 in. deep and 2 ft. wide and weighing up to 34 tons, were delivered whole to the site. On arrival, they were lifted over the outer girder of the existing bridge by two 36-ton breakdown cranes, landed on bogies on the trestles and rolled out into their required positions. The cross-girders were placed in position and riveted, and the deck concreted, waterproofed and tiled.

The weight of the new span during the rolling-in process was 440 tons. This was taken on six carriages, the arrangement of one of these together with the track and balls being shown in Fig. 19. It will be seen that the tracks are marked out in feet, which assists the gangs at either end of the span to keep the bridge

Fig. 16.—New Cross Bridge. Trestle on towpath

parallel to its final position whilst it is being moved. Two hand winches were used to move the span.

To prepare the old bridge for rolling out, each lattice girder was stiffened over the trestles by heavy vertical timbers, after which it was jacked up off its bearings and the carriages introduced. The tops of the existing

Fig. 17.—New Cross Bridge. Piled trestle in canal

piers were reconstructed as shown in Fig. 18 to allow for drainage, and ease of access to the new bearings.

On a selected Sunday, the bridge was closed to rail traffic, and after removing the tracks, the old bridge was pulled out sideways and the new one pulled in. The tracks were reinstated temporarily on timbers on the new bridge and electrification and signalling restored. Fig. 20 shows the new bridge being rolled in during that day. The timbers shown across the top flanges of two girders support cables which could not be broken, and which were originally supported on the old bridge. The new span was passed under the cables, which were

Fig. 18.—New Cross Bridge. Shows preparations for access to bearings of new bridge and drainage

supported on rollers, the timbers being moved over as required.

The new bridge was rolled in at a level higher than its final level, and by means of hydraulic jacks and brackets fixed to the ends of the girders, was slightly raised to enable its rolling carriages to be removed, then lowered on to its bearings. At a later date, ballast was unloaded on the bridge and the permanent ballasted track completed.

Fig. 19.—New Cross Bridge. Roller path, carriage and balls

To give access to the underside of the completed bridge for painting and maintenance, a travelling cradle was mounted on the lower flange of the outer girders. It was stabled at the south end of the bridge, the trestle piles being cut off at a suitable level and fitted with fenders to protect both the pier and cradle from damage by canal craft.

The existing bridge was demolished by cutting out the floor system in suitable sections and lowering into barges. The main girders were rolled over to the new bridge one by one, and supported from it whilst they were burnt up in sections for disposal in the same way.

Galena Road Bridge, Hammersmith

Although not a large bridge, this is an interesting example of one which was jacked up into position.

The old bridge consisted of two separate spans side by side, each carrying two tracks, and having three main

girders, with joist cross girders and longitudinal timber track bearers and decking. The bridge was weak under modern traffic and deflections of the main girders and decking were large. A number of the cross girders had fractured close to or at their end connections due to fatigue, and had been repaired from time to time.

Fig. 21 shows the arrangement of the new bridge, with the old one superimposed in dotted lines.

It was found permissible to reduce the span by erecting columns at the edges of the paths, and this also allowed a ballasted deck, which was considered desirable. It should be mentioned that the bridge is over a minor

Fig. 20.—New Cross Bridge. Old bridge on the left has been rolled out and new bridge is being rolled in

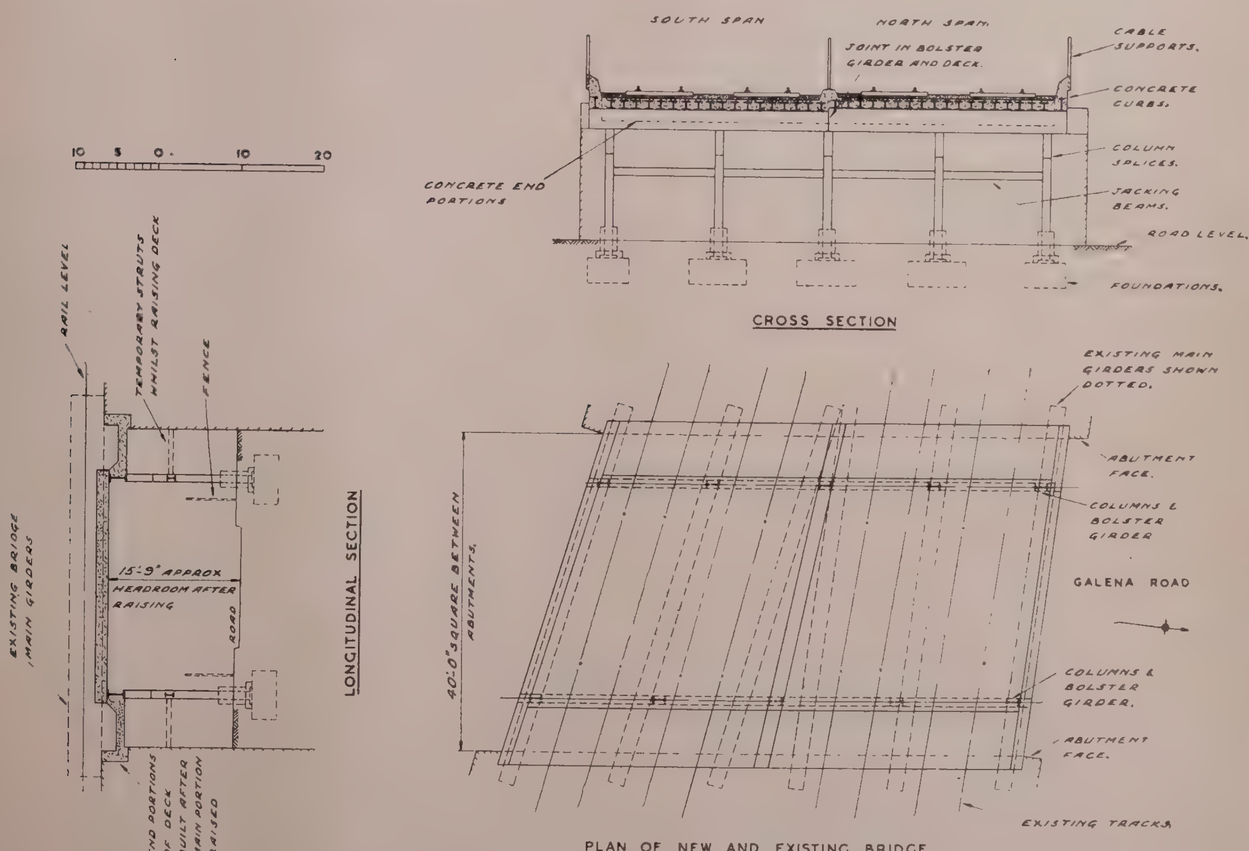

Fig. 21.—Galena Road Bridge. General arrangement

Considerable corrosion had taken place in the bottom flanges of the main girders and under the timbers on the cross girders.

One reason for the unusual design of the new bridge was the desire to improve the alignment of the tracks. To do this, it was necessary to eliminate the centre main girders, and construction depth being very restricted, an alternative method was sought.

residential road varying in width from 18 ft. to 22 ft., with very little through traffic.

The new deck consists of broad flanged beams encased in concrete above their bottom flange level and supported on transverse bolster girders situated approximately 6 feet from each abutment.

The method of erection is given in Fig. 22. The deck was built in halves, so that only two tracks would be

affected at a time. Stage 1 shows one half ready for raising the first 1 ft. 4 in., the decking beams having been concreted with a working clearance of 1 ft. 8 in., and the waterproofing and tiling completed. Four jacks, each of forty tons capacity, are in position on the trestles.

approximately 1 ft. 8 in. The temporary column extensions were replaced by the permanent ones, and the deck finally lowered on the column caps and connected. Relaying of the tracks on ballast completed the main work.

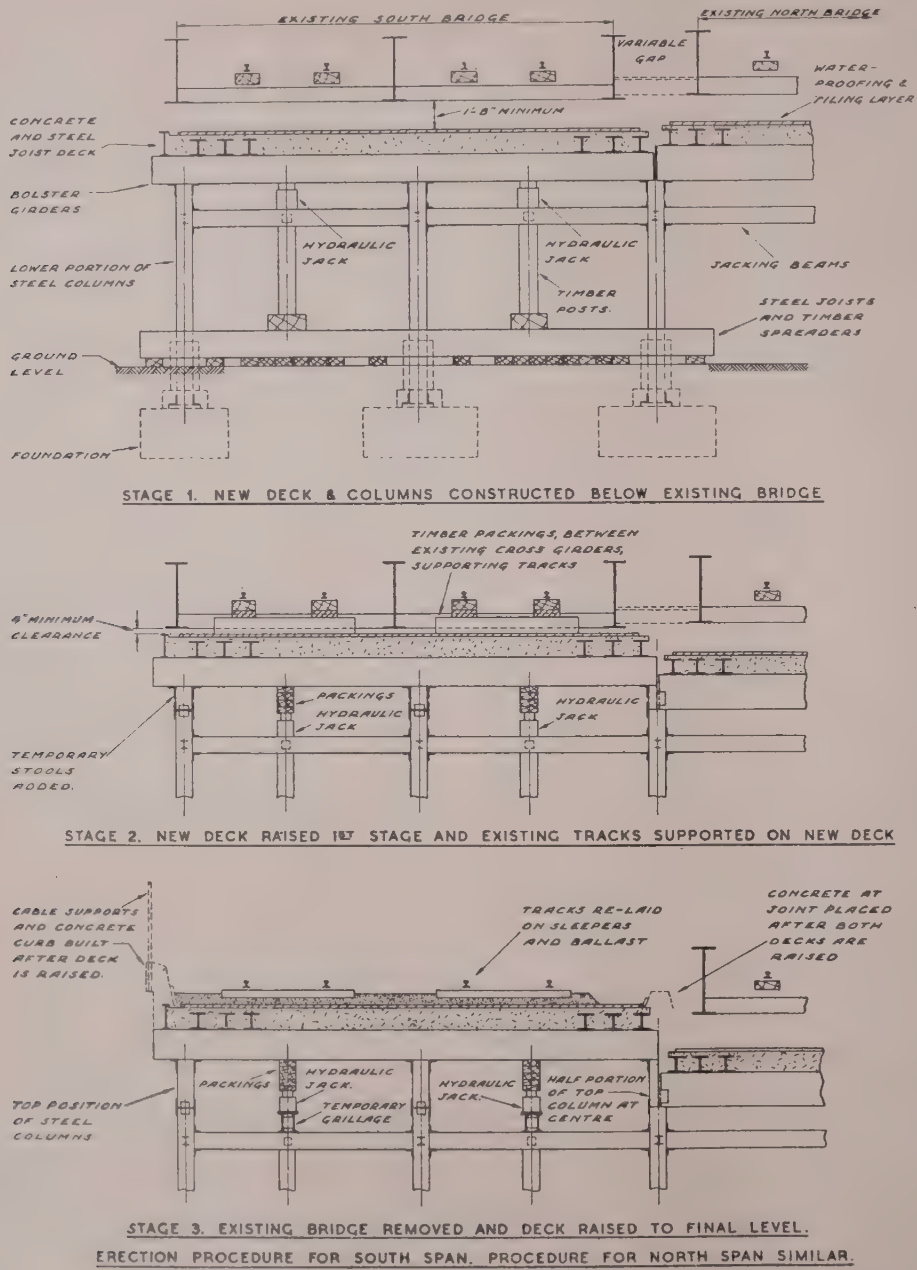

Fig. 22.—Galena Road Bridge. Lifting operations

Particular care was taken during lifting to ensure that the four independently worked jacks lifted vertically and were extended the same amount in a given interval of time so as to avoid the possibility of the deck wandering in plan from its correct position.

Stage 2 shows the first lift completed and the temporary column extensions added. At this stage, the existing tracks were packed from the new deck, which enabled the existing main girders and cross girders to [illegible]

The jacks were then re-seated at a higher level, and during [illegible] the two existing tracks were removed and the half deck raised its final amount of

The foregoing procedure was repeated for the second half of the bridge.

Acknowledgements

Grateful acknowledgements are due to Mr. F. E. Campion, Civil Engineer, British Railways, Southern Region, for permission to describe Stewarts Road Bridge and the Twickenham Flyover, and to Mr. C. E. Dunton, Chief Civil Engineer, London Transport Executive, in the case of the New Cross and Galena Road Bridges.

The author's firm, Joseph Westwood & Co., Ltd., of Millwall, London, was the main contractor for the four bridges described.

A Welded 100-ft. Aluminium Tower

By Cedric Marsh, B.A.(Graduate)

Summary

This account covers the design, fabrication and testing of a tubular aluminium tower used to carry lighting projectors for the illumination of a marshalling yard at Bienne, Switzerland. It is concerned mainly with those problems encountered in the design and welding of aluminium structures and the behaviour of the erected tower.

Introduction

During the development of the use of aluminium in structural engineering attention has been focused mainly on the use of extrusions. This is in no way remarkable as extrusions possess the very desirable characteristic of versatility and permit the design of sections of high structural efficiency by the variation of wall thicknesses and the introduction of lips and bulbs. By now most of the possible variations on the basic structural sections must have been studied and the development and standardisation of structural sections suited to the characteristic of aluminium is almost completed.

The use of sheet aluminium in structures, however, has not received quite the same amount of attention but there is a trend on the Continent to consider more seriously this structural medium because of the various advantages it offers.

Sheet or plate is, in general, cheaper than extrusions, the actual difference varying from country to country, and even when formed into a structural section it can be cheaper than the extruded one. Although it is impossible to vary the thickness to gain efficiency it is possible to have much higher ratios of overall dimensions to wall thickness and to form larger sections than are at present possible in extrusions. By utilising relatively thin walls local buckling provides the design criterion and, as this is related to elasticity rather than tensile strength, little benefit is gained from using high strength alloys, thus allowing the use of those of medium or low strengths which are cheaper. Because local buckling governs stress is often low and welding, which reduces strength in all but the annealed alloys, can be utilised to advantage with less regard to the lowering of the strength than is usually paid.

The tower with which we are concerned is a simple tapered tube of 100 ft. in height welded up from aluminium sheet. (Fig. 1.)

In the past, goods yards have been illuminated by numerous low power (400 watt) lamps on standards of some 30 ft. in height. This system, although possessing considerable adaptability, creates dark spots, requires extensive wiring and cleaning of the lamps is frequent. In France a system has been developed using a few high towers carrying high power projectors, up to 2000 watts, which flood the yards with light. Because of the height, the reflectors are not affected by fumes from trains and the wiring is simplified.

Such towers are normally of latticed steel construction using either angles or tubes. This type of tower offers serious disadvantages in that the ladder providing access to the projectors is open to the weather and climbing is dangerous in high winds or when the ladders are iced, the risk of vertigo and falling is always present, also unauthorised persons have access to the tower. The value of a closed tower is thus immediately evident while, by the use of aluminium, maintenance is virtually eliminated and erection and any future striking and re-erection are facilitated.

The erected cost of the aluminium tower and foundation was no higher than that for the installation with a steel tower, but it must be borne in mind that the ruling market conditions in Switzerland are not identical with those in the U.K.

Dimension and Equipment

The tower height, 100 ft., is dictated by the fact that the projectors must be above the influence of fumes from engines and high enough to command a wide range of the yards, but not so high that the intensity of light at ground level would be seriously reduced. The base dimensions 4 ft. 6 in. was such that the tower could be fitted between existing tracks and not interfere with the layout of the yard while the tapered form was adopted as a natural one (although the latticed steel towers are parallel), the top diameter being 2 ft. 3 in. to allow the passage of a man.

Fig. 1.—The erected tower

Sheet lengths of 10 ft., of 4 ft. 6 in. in width, were the maximum available or that could be formed in the available roll formers. Thus the lower sections were welded up from three curved sheets, the upper sections from two sheets, to make up tubular segments of 10 ft. in

length. These sections were then welded together with an H-section extruded stiffening ring (Fig. 2) between them to form shop fabricated sections of a maximum length of 30 ft. No longitudinal stiffeners were used but stiffening rings were felt to be advisable. They also facilitated fixing the internal equipment.

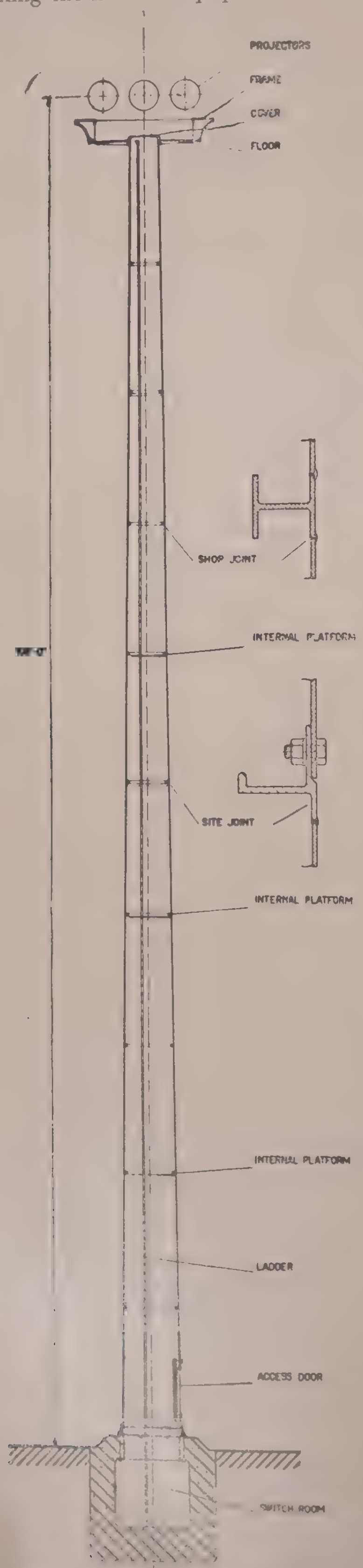

Fig. 2.—General details of the assembled tower

The sheet thickness, 0,22 in. at the base, was reduced 0,02 in. every 10 ft. to a minimum of 0,12 in. for the upper 50 ft. This minimum thickness was governed by the fact that the tower was to be welded as thinner material requires welders of considerable experience.

The platform surmounting the tower (Fig. 3) was proportioned to be of some æsthetic value. The ratio of the sides are those of the "golden" rectangle, $1 : \sqrt{2}$, 7 ft. 10 in. × 11 ft., with the sides curved. Provision for the fitting of 8, 2000 watt, projectors is made. Lighting immediately round the tower can be effected

Fig. 3.—View of projector platform

by a lamp fitted at 50 ft. up. A continuous aluminium ladder runs vertically up the inside of the tower with internal platforms every 20 ft. up to 60 ft. and on the top of the tube is fitted a lid to keep out the weather.

Structural Alloy

Where extrusions are used it is now widely accepted that for structural work the heat-treated Aluminium-Magnesium-Silicon alloy designated HEIOWP is the most suitable. This alloy has a proof stress of 15 ton/sq. in. and an ultimate strength of 18 ton/sq. in. In the welded condition, however, the strengths fall to 7 ton/sq. in. and 12 ton/sq. in. respectively.

In the present structure the work-hardened aluminium $2\frac{1}{2}$ per cent. magnesium alloy NS4½H was used. This alloy is one of the most resistant to corrosive attack and possesses the following minimum properties.

	original plate	as welded
u.t.s.	15 ton/sq. in.	11 ton/sq. in.
0.1 per cent. proof stress	12 ton/sq. in.	6 ton/sq. in.
quoted fatigue strength (50×10^6 cycles) ...	6 ton/sq. in.	5 ton/sq. in.

For extrusions NE4 was employed which possesses the same as-welded properties as NS4.

The adoption of an alloy with these relatively low properties, although not greatly inferior to HEIOWP in the welded condition, is justified because of the ruling consideration of deflection while, because of the high ratio of radius to thickness, local buckling in the tower walls would occur at a fairly low stress, thus the tensile strength of the material is not of great importance.

It is worth remarking that the transverse weld runs are immediately adjacent to stiffening rings and thus the buckling stress will only be related to the full strength of the plate.

Fig. 4 shows the straight line type of strut curve used to establish the critical buckling stress. The straight line portion of this curve is given by the expression :

$$p_{cr} = p_y\,(B - D\lambda) \qquad (1)$$

where p_{cr} = buckling stress, p_y = 0.1 per cent. proof stress, B and D (constants related to the proof stress) = 1,13 and 0,0075 respectively and λ = the equivalent slenderness ratio of strut or wall :

Thus : $p_{cr} = 27000\,(1{,}13 - 0{,}0075\lambda)$

At the upper end the line is cut off at the proof stress and at the lower end the curves become the Euler stress :

$$p_{cr} = \frac{\pi^2 E}{\lambda^2} \quad \ldots\ldots\ldots\ldots \quad (2)$$

This type of strut curve is adopted, rather than the Perry type curve, because of the greater facility by which it can be introduced in analyses to determine the most efficient proportions of a section when buckling governs.

It was expected that the tower would oscillate in the wind and for this reason some attention was paid to the fatigue strength of the welds. The design allowed a factor of safety of 2 on the quoted fatigue strength. This was not considered too high as a laboratory figure bears little relationship to practice owing to the perfection of laboratory test pieces.

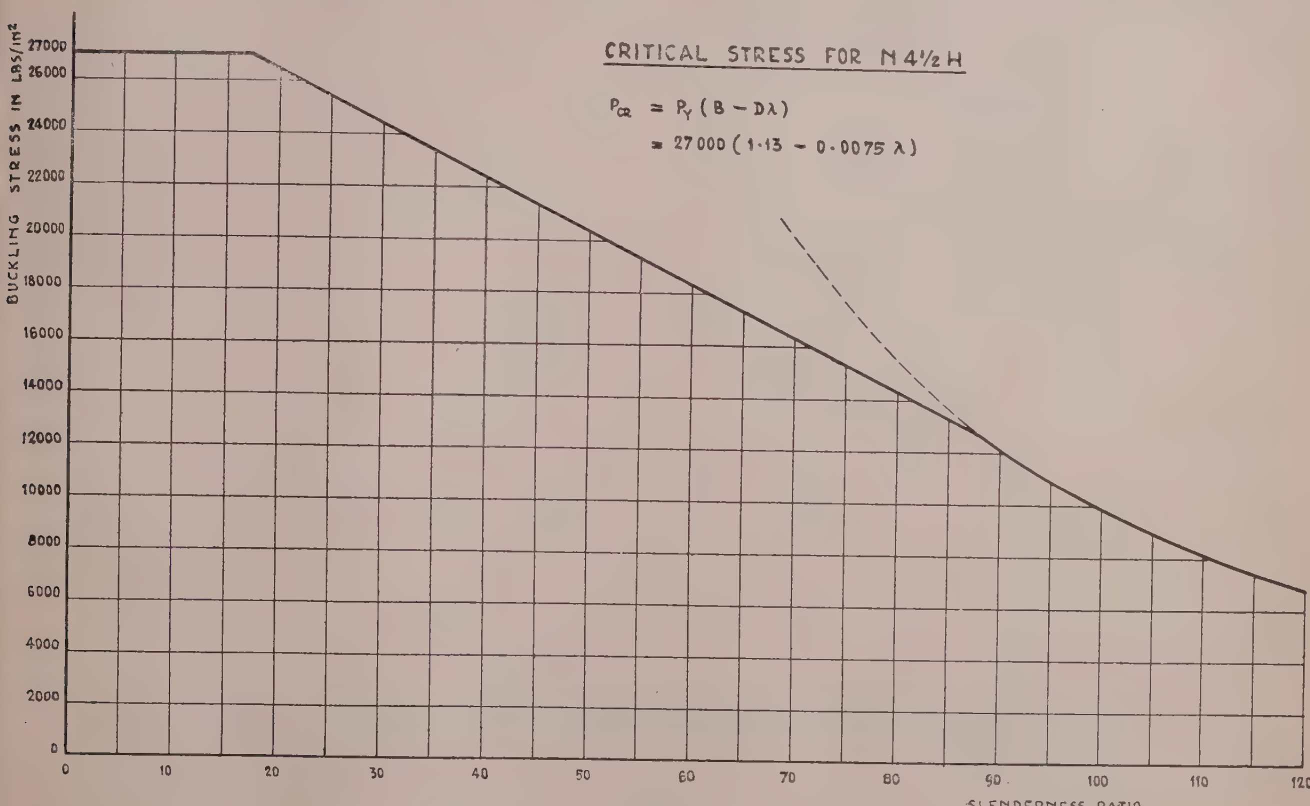

Fig. 4.—Curve of straight line strut formula

Welding Aluminium

Although few authoritative specifications yet deal with the application of welding in aluminium structures, there is a growing interest in this very desirable method of jointing and a growing number of examples of its use.

The former objections to welding were that it was necessary to use fluxes with the attendant risk of corrosion if inadequately washed after welding. Also that gas welding, although capable of producing good welds with certain alloys, tends to give an unduly wide heat affected or softened zone adjacent to the weld, particularly when dealing with heat-treated or work-hardened alloys.

The first of these objections has been overcome by the development of the argon tungsten arc and the inert gas shielded metal arc systems of welding in both of which a flow of argon gas is used to shield the weld pool to prevent oxidation thereby eliminating the need for flux, the removal and corrosive nature of which had earlier provided serious problems. The loss of strength cannot be prevented, although these modern welding methods, because of their speed, greatly reduced the annealed zone, and the design must be conducted taking this reduction into consideration. It is an undesirable feature but by intelligent design its influence can be virtually eliminated.

In the fabrication of the lighting tower the argon tungsten arc system was used throughout. This method uses a non-consumable tungsten electrode, shielded by argon. An arc is struck between the electrode and the work, and filler rod, in our case of the same alloy as the sheet, is fed into the side of the weld pool.

Because of the speed of welding in aluminium, twice that with steel, it is advisable to train welders, preferably from scratch, before they have become too hardened to the ways of steel welding. In the present case the firm responsible for the fabrication of the tower had no previous experience in the welding of aluminium and to ensure that this lack of knowledge had no detrimental effect on the finished product a full and continuous test programme was conducted before and during fabrication to ensure welds of the desired quality.

Before any fabrication was started, replicas of parts of the tower were welded up by the newly trained welders and subjected to visual examination and mechanical tests. The original trials, as expected, proved to be well below standard, inclusions and porosity being very evident. The faults were shown to be due to various

causes including an unsuitable high frequency unit, unclean argon gas, and welding too slowly. Modification of these factors, the introduction of a grooved aluminium backing strip and the use of single-V edge preparation resulted in consistently good welds, free from inclusions or porosity, with the following properties :

u.t.s.	12 tons/sq. in.
0.1% proof stress	7 tons/sq. in.
Fatigue limit (10^6 ⌐) ...	4 tons/sq. in.

It was originally intended to jig the parts so that the longitudinal edges to be joined were held rigidly in relationship to each other. During the first trial with this rig, owing to the high heat in-put and high coefficient of expansion of aluminium, the edges buckled and the relative displacement was such as to make welding impossible. The rigid jig was therefore dispensed with and the ends of the seams simply clamped together with a slight taper in the gap between the edges which closed up as welding progressed. A backing strip held manually behind the seam allowed a fluid weld pool and gave a clean bead on the reversed side.

Fig. 5.—X-ray unit for weld examination

The tubes were mounted to permit the turning of the assembly in order to allow down hand welding throughout.

Owing to the height of the tower, the wind velocity is greater at the top than at the base and the unit wind pressure assumed, as given in the relevant specifications, varies from 14 lb./sq. ft. at the base to 20 lb./sq. ft. at the top corresponding to a maximum wind velocity of 86 m.p.h. as computed from the formula :

$$p = \frac{\gamma}{2g}v^2 \quad \ldots \ldots \ldots \ldots \quad (3)$$

where p = pressure, lb./ft.^{2}, γ = density of air, 0,08 lb,/ft.^{3}, g = 32 ft./sec.^{2}, v = wind velocity, ft./sec.

This unit pressure is then multiplied by various factors to give the actual applied loads. For the tower itself, being round and smooth, the factor is 0,55. Fig. 7 shows the value of the loads for which the tower was finally designed.

Experience has shown that if the beams of light from the projectors are to be kept reasonably stable the deflection of a lighting tower should not exceed 1/75th of the height, giving, in our case, an allowable deflection of some 16 in. This consideration provided the design criterion.

Observation of the behaviour of the tower under wind load would indicate that for a given wind velocity the theoretical load on the tower is too high.

Design

It is essential in the design of aluminium structures that the utmost is done to conserve material. The relative prices of steel and aluminium are such that in normal structures a weight ratio of 5 : 1 is usually necessary if direct competition is to be possible. In the present case, because of the nature of the work, the simplicity of fabrication and the fact that the steel had to be galvanised, a weight reduction of 40 per cent. of the steel weight was sufficient. In view of the fact that deflection governed, aluminium having only 1/3rd the elastic modulus of steel, and that the base dimension was limited, the problem seemed insurmountable but by a fully rational disposition of the metal in the tower the necessary low weight was realised.

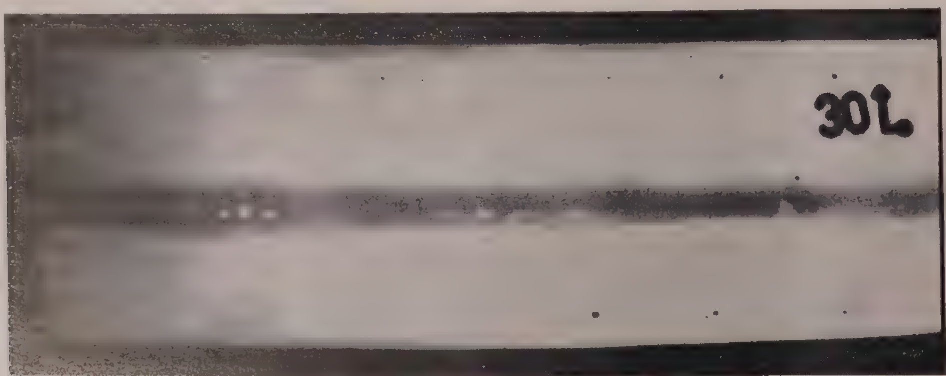

Fig. 6.—Weld showing porosity

On completion of the tower all suspect welds were examined by X-rays, the unit shown in Fig. 5 being used. Those welds showing inclusions or porosity as exemplified in Fig. 6 were cut out and re-welded.

Welding of the platform provided some difficulty due to distortion and some parts were finally riveted.

Design Loading

The only load carried by the tower, other than dead weight, is wind load and the entire design is related only to wind forces.

Although, in this instance, deflection was of an overriding importance, it is worth discussing the efficiency of tubes in bending.

As previously mentioned in thin-walled tubes elastic instability of the walls occurs before the proof stress of the material is reached, thus, if a given area of metal is formed into a tube, as the radius, R, is increased, the wall thickness, t, becoming less, the modulus of the section increases in linear proportion to R. The buckling stress, however, falls and at some ratio of R to t the

section will be of maximum strength beyond which an increase in R will reduce the moment of resistance.

The buckling stress, established experimentally, for a curved sheet is given by :

$$p_{cr} = 0.3E \frac{t}{R}$$

Equating this to the Euler formula for a strut :

$$\frac{\pi^2 E}{\lambda^2} = 0.3E \frac{t}{R}$$

gives an "equivalent slenderness ratio" for the tube wall of :

$$\lambda = 5.7 \sqrt{\frac{R}{t}} \quad . \quad . \quad . \quad . \quad . \quad (4)$$

By referring this slenderness ratio to the strut curve, Fig. 4, the buckling stress of the wall is given immediately.

This method is adopted as the most easily applied of all methods which takes into account automatically the transition from material breakdown to purely elastic failure as λ increases. The use of the exact formula for elastic buckling is considered to be too optimistic while the use of a factor based on the reduced modulus in the elasto-plastic range is too clumsy.

By adopting the straight line strut formula (1) page 334, it is perfectly simple to show that the tube of maximum efficiency is one in which the equivalent slenderness ratio of the walls is :

$$\lambda = \frac{B}{2D}$$

which in our case, gives $\lambda = 75$ and $\frac{R}{t} = 172$. This figure is somewhat higher than can normally be adopted, but the maximum value of $\frac{R}{t}$ in the tower is actually 174, the value at the base being 127.

To proportion the tube to use the minimum amount of material for the specified deflection provides a rather more difficult problem. Mathematical analysis is possible to a certain degree but as the load on the tube varies along the length and is not a continuous function of the diameter no satisfactory result was obtained. It was finally decided to vary the sheet thickness to give a reasonably constant stress along the tower, an arrangement which gave a high degree of flexural efficiency. Analysis of variations of this distribution of metal showed that no further worthwhile economy could be made.

The simplicity of the form makes for a simple analysis. Bending stresses were computed directly and are shown in Fig. 7. The deflection was calculated by the expression :

$$\delta = \Sigma \frac{Mxs}{EI}$$

where M is the bending moment at the centre of a 10-ft. segment, x is the distance from the top, and s is the length of segment, 10 ft. This calculation gave a value of 17 in., which is slightly higher than the specified value but was accepted.

After erection of the tower, it was subjected to a pull at a point 60 ft. up via the gin pole seen in Fig. 11. For a load of 1900 lb. the deflection at the top of $5\frac{1}{2}$ in. equalled, within measurable error, the calculated deflection.

Because of the flexible nature of the tower and because of its smooth tubular form, it was necessary that the possibility of forced vibrations by the wind should be studied.

To calculate the natural frequency of the tower it is assumed to be in a horizontal position and the deflection

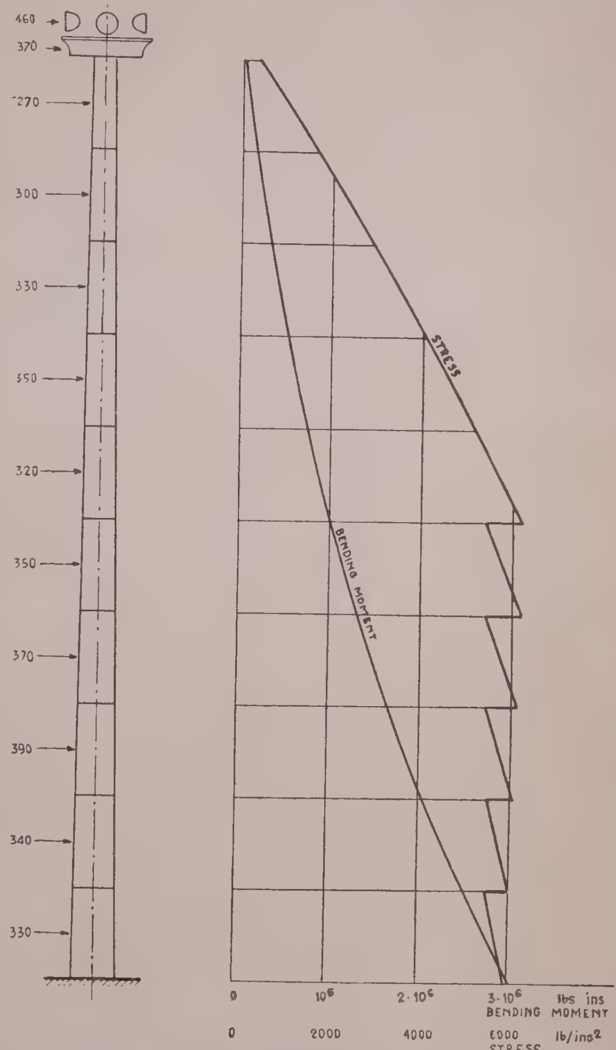

Fig. 7.—Loading, bending moment stress

Δ, due to dead load, computed. The natural frequency of the oscillation is then given approximately by :

$$f = \frac{1}{2\pi} \sqrt{\frac{g}{\Delta}}$$

This gave a frequency, with 8 projectors in position, of 0.64 ∿/sec. By suddenly releasing the load, applied in the deflection test, it was possible to obtain the true frequency of 0.76 ∿/sec. which would indicate an overestimate of the weight as agreement in the deflection test showed that the elastic modulus and sheet dimensions were as in the calculations. This proved to be the case, the assumed weight of the projectors being high, but even with the correct weight the analysis gave 0.68 ∿/sec. which is still 10 per cent. in error.

Knowing the natural frequency it is then necessary to establish the frequency of the eddies shed from the

alternate sides of the tower which are created by a flow of air past the tower and can set up a forced vibration.

Theoretically the frequency of the impulses is given by:

$$f = 0.2 \frac{v}{D}$$

Where v is the wind velocity in ft/sec. and D is the tube diameter in feet. As the diameter of the tower is not uniform the frequency of impulses will vary along the length. At a point three-quarters the way up the tower at a wind velocity of 10.7 ft./sec. (7.3 m/h) the wind impulses will synchronise with the natural frequency.

When the eddie frequency is resonant with the tower the energy input by the wind is given approximately by :

$$G = 0.000022\ V^2\ D\ A \text{ lb. ft./cycle/ft. length.}$$

where V = wind velocity, m/hr, D = diameter, ins., A = amplitude, ins.

Assuming a sinusoidal curve for the deflected tower, we obtain, by integrating the above expression over the height of the tower.

$$G = 0.055\ V^2 \delta, \text{ where } \delta = \text{the deflection at the top.}$$

The energy absorbed by the tower cannot be calculated from initial data and has to be obtained from direct tests on the tower itself. When the tower is allowed to vibrate freely, the loss of energy in each cycle is given by the loss of strain energy at extreme deflection, in successive cycles. The strain energy is assumed to be proportional to the square of the deflection and is calculated initially for the case of deflection due to dead weight. From the measured damping of the tower during the vibration tests the following expression was obtained for the energy lost by damping :

$$E = 4.5\,\delta^2 (1 - e^{-0.2}) = 0.81\,\delta^2 \text{ lb. ft.}$$

When the eddies and the tower have a common frequency the energy input must equal the energy absorbed in damping and the steady amplitude of the tower is given by :

$$G = E$$

For a velocity of 7.3 m.p.h., $G = 2.9\delta$ Therefore :

$$\delta = 3.6 \text{ in.}$$

i.e., total amplitude = 7.2 in.

We thus see that for wind velocities at which resonant forced vibrations are probable the energy input is too small to cause serious swaying of the tower or to cause repeated stresses of an order capable of causing fatigue failure.

It may be mentioned here that these oscillations occur across the wind stream and are thus independent of the deflection due to direct wind load.

Early observations showed no tendency for the tower to vibrate and, although no reliable estimates of the wind speeds could be obtained from local observers, it was concluded that oscillations would not occur. However, after several months, the first steady breeze arose and the tower oscillated through 12 in. As the wind velocity mounted the vibrations stopped and the tower swayed in the direction of the wind, as is to be expected. Observers' estimates of the wind speed varied from [illegible] 60 m.p.h., it was decided to correlate exactly the wind speed and the response of the tower using an anemometer to measure wind velocity and a scale on the top of the tower viewed through a theodolite to measure the amplitude of the vibration.

The large amplitude of the tower was discovered to be attributable to incorrectly tightened bolts and on correcting this the observed oscillation showed a maximum total amplitude of 4.5 in. for a range of wind speeds between 6 and 10 m.p.h. and 5 in. for a range between 40 and 45 m.p.h. The latter range is due to a frequency of wind eddies equal to the fifth harmonic of the natural frequency.

It is seen that the predicted range of dangerous velocities agrees reasonably well with that observed but that the predicted steady amplitude was too high indicating an inaccuracy in the expression for the energy input due to wind. This however is on the safe side.

Although the maximum amplitude attained still only creates a low stress it was felt that some effort to curb these oscillations should be made. The simple expedient of introducing additional weight into the tower in form of a few steel plates on the upper platform reduced the maximum observed amplitude to 2 in. and unless further stiff breezes occur which disproves the efficacy of this device no further steps will be taken.

Foundation

With the usual lighting tower of the latticed steel type, a separate hut is erected to house the necessary electrical equipment, the cost of this hut being as much as a third that of the tower itself. In order to save this cost it was decided to adopt a hollow foundation in which this equipment could be installed, access being via the door at the base of the tower through a trap-door in the floor and down a short ladder.

Because of the enclosed nature of the tower this arrangement provided no waterproofing problems, while offering some advantages over a solid concrete foundation.

As the base is relatively deep, overturning of the tower is resisted by pressure on the sides of the foundation as well as on the bottom thus by increasing the foundation size by hollowing the block, it is possible to obtain the same resisting moment for a given maximum ground pressure while actually economising in concrete.

In order not to overestimate the safe pressure on the soil at ground level, as this soil would probably be disturbed, a parabolic pressure distribution, Fig. 8, was

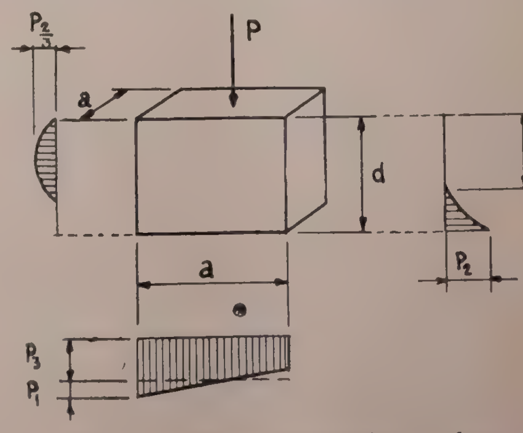

Fig. 8.—Pressure distribution on base

assumed in the analysis for the pressure on the sides of the foundation block. The pressure distribution across the base varies linearly :

The maximum ground pressure is then :

$$p = \frac{12\,b\,M}{a(d^3 + 4a^3)} + \frac{P}{a^2}$$

In our case we have : $P = 54$ tons $M = 127$ tons/ft.
$a = 10$ ft. 6 in. $d = 9$ ft. 3 in.
giving $p = 0.77$ tons/ft.^{2}.

Because of the hollow type of foundation the walls are subjected to bending moment due to the ground pressure while the edges of the foundation floor carry shear stress. This required the introduction of more reinforcement than in a normal foundation block but not excessively so.

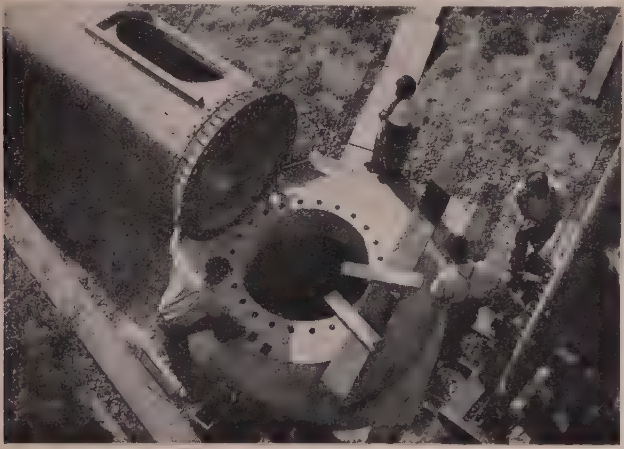

Fig. 9.—View of hollow foundation and tower base

A galvanised steel base ring, as seen in Fig. 9, provided the fixing between the tower and the foundation. Galvanising itself was not considered to be a sufficiently effective protection for the aluminium/steel contact and for added security a zinc chromate jointing compound was used to fill completely the crevice between the faying surfaces thus excluding any possibility of trapped water promoting corrosion.

Cadmium plated steel bolts ($\frac{5}{8}$ in. Φ) fixed the base ring to the tower. To fix the base ring to the foundation, long bolts ($1\frac{1}{4}$ in. Φ) passed through the roof of the base block, with the bolt heads on the underside.

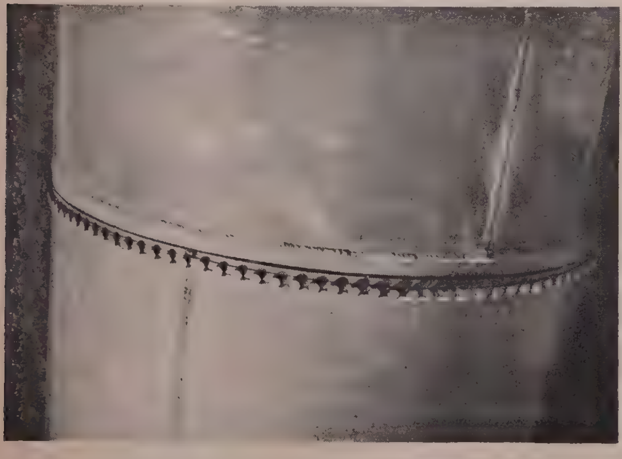

Fig. 10.—Site joint with locking strip

Erection

The shop in which the tower was fabricated is little more than a mile from the site and transport of the 30 ft. lengths of tower provided no problem. On the site the separate lengths were assembled on the ground and bolted together using aluminium bolts. To provide a locking device for the bolts, in case wind induced oscillations of the tower should tend to loosen them, the aluminium strip seen in Fig. 10 was introduced and hammered over between the nuts.

The platform was actually welded to the top of the tower in the shops. Other parts such as ladders, internal platform, the doors and cover were fitted either in the shop or on site as proved most convenient.

Incorporated in the welded steel base ring are two brackets which in conjunction with brackets cast into the top of the concrete base, form a hinge when a steel pin is passed through them. The tower fully assembled was then pulled up, Fig. 11, by a wire rope fixed to a steel collar, at a point 60 ft. along the tower. A simple hand-operated winch was used for this purpose, the lifting requiring some 20 minutes. Hand-controlled guy ropes prevented any lateral sway of the tower during lifting. When erect the final adjustment of the base was made by wooden wedges under the base ring followed by grouting in the bolts.

Behaviour of the Tower

Apart from those actions of the tower discussed above which, if not wholly foreseen, had been considered as possible, others, quite unexpected, also occurred.

Fig. 11.—Lifting the tower

Immediately after erection at noon on a relatively sunny day, it was observed that the tower was not upright, the top being 12 in. out. As the sun went behind a cloud the tower slowly recovered its true position. The deflection was caused by the sun heating one side of the tower which expanded. A deflection of some 12 in. however seemed excessive and calculations showed that this required a temperature difference of some 55°F. between the opposite sides of the tower. Measurements confirmed that this temperature difference actually arose, in spite of the high conductivity of aluminium. Only negligible stresses are created during this deformation and as the towers are in use only at night the effect never influences the beams of light.

After the installation of the electrical equipment, again on a sunny day, rather disturbing groans and bangs emanated from the tower and at noon the walls of the tower were seen to have buckled slightly. Again expansion was the cause. To carry the cable up to the projectors, a timber board had been fixed to the inside of the tower. On expanding, the aluminium was thrown in compression while the timber carried tension, the noise deriving from the slipping of bolts. Again calculations showed that this was quite to be expected and on allowing movable fixings the buckles disappeared with no harm done to the walls as the buckles had been purely elastic.

Three of these towers have been built but all the references to behaviour relate only to one tower. The

sway due to expansion is common to them all but only in one was there buckling due to expansion and only one tower, the same one, oscillated in the wind. In all respects the other two towers have behaved impeccably.

Conclusions

Early work on the use of aluminium in structures leant towards the use of high strength alloys with either a bias towards aircraft design or simple steel design. None of these views are now held, high strength alloys being subject to corrosion, aircraft type structures being too expensive in fabrication and steel type structures too expensive in material. The tower we have described represents an extreme in the opposite direction. A low strength alloy is used and a very simple structural form is adopted which has no relation to earlier steel structures in this application.

Economically, the structure is fully justified, being actually cheaper than the schemes employing steel, while the advantage of durability without maintenance and an æsthetic form provide an immediate appeal.

The practicability of welding in aluminium structures is convincingly illustrated, particularly where deflection is the governing factor, but it is also seen that lack of familiarity with the application of this method of jointing still provides a slight handicap to its wider acceptance and use, a handicap which, we are glad to say, is rapidly being reduced.

This tower also shows that in certain cases aluminium can be a competitive structural material, and, with the development of structural forms suited to the characteristics of these alloys, the number of such structures will undoubtedly increase.

Aluminium, with only a few decades of real study of its use as a structural material, still provides some problems and can behave in some unforeseen ways, but the body of experience that has now been accumulated is responsible for a high degree of confidence in its application in the sphere of structures.

Acknowledgements

The realisation of this lighting tower has been the result of full and continuous co-operation between the Swiss Federal Railways, who laid down their requirements, Aluminium Laboratories Limited, Geneva, who determined the form the installation should take and carried out the appropriate analysis, Hartmann & Co., of Bienne, who fabricated the tower, Aluminiumwerke A.G. Rorschach who supplied the material and provided technical assistance on the fabrication problems, and Prof. Paschoud, of the University of Lausanne, who conducted the laboratory and field tests.

Institution Notices and Proceedings

GENERAL MEETING

A General Meeting of the Institution of Structural Engineers was held at 11, Upper Belgrave Street, London, S.W.1, on Thursday, October 7th, 1954, at 6 p.m., when the Presidential Address for the Session 1954-55 was given by Dr. S. B. Hamilton, M.Sc., Ph.D., B.Sc.(Eng.), A.R.C.S., M.I.C.E., M.I.Struct.E. The Retiring President, Lt.-Colonel R. F. Galbraith, M.C., B.Sc., M.I.Struct.E., A.M.I.C.E., was in the Chair.

The Chairman welcomed the guests who were present and then presented the following award :—

Drury Medal, 1953, to Mr. G. W. E. Feakes (Student).

After some introductory remarks regarding Dr. Hamilton's career in connection with the Institution, the Chairman invested him with the Presidential Badge.

Dr. Hamilton then took the Chair and called upon Mr. L. Scott White, O.B.E. (Past President), to propose, and Mr. J. Guthrie Brown (Vice-President) to second a vote of thanks to Colonel Galbraith for his work as President of the Institution during the Session 1953-54.

The vote of thanks was carried and Colonel Galbraith responded briefly.

Dr. Hamilton then gave the Presidential Address for the Session 1954-55, which is printed in this issue.

At the conclusion of the Address, a vote of thanks to the President was proposed by Mr. L. E. Kent (Vice-President) and seconded by Mr. S. Vaughan (Vice-President). This was carried with acclamation and the proceedings then terminated.

ORDINARY GENERAL MEETING

An Ordinary General Meeting of the Institution of Structural Engineers was held at 11, Upper Belgrave Street, London, S.W.1, on Thursday, October 28th, 1954, at 5.55 p.m., Dr. S. B. Hamilton, M.Sc., Ph.D., B.Sc.(Eng.), A.R.C.S., M.I.C.E., M.I.Struct.E. (President), in the Chair.

The Minutes of the Ordinary General Meetings held on May 27th and June 24th, 1954, as published in the Journal, were taken as read, were confirmed and signed.

The following members were elected in accordance with the Bye-Laws. Will members kindly note that the elections as tabulated below should be referred to when consulting the Year Book for evidence of membership.

Graduates

Baikoff, Eugene Marc Alexander, of Bergvlei, Transvaal, South Africa.
Bhar, Gurdarshan Singh, of London.
Cartlidge, Charles Peter, of Gainsborough, Lincs.
Crisp, Henry George, of Culcheth, nr. Warrington, Lancs.
David, Evan Savours, of Bristol.
Day, Michael Chester, of Benoni, Transvaal, South Africa.
Ejsymont, Zbigniew, of New Malden, Surrey.
Farrow, Thomas Henry, of London.
Gabay, Leslie Oliver St. Clear, of St. Andrew, Jamaica, B.W.I.
Gould, Harold, of Lusaka, Northern Rhodesia.
Hanks, John Wilfred, of Lambton, Germiston, Transvaal, South Africa.
Henderson, Leo Gordon, of Cooma, N.S.W., Australia.
Hughes, John William, of Sydney, N.S.W., Australia.
Husain, Mahmood, of London.
Kuczynski, Edward, of London.
Nurse, Colin Frederick, of Scunthorpe, Lincs.
Patrick, Colin Desmond, of Johannesburg, South Africa.
Pereira, Oscar Joseph, of London.
Stefanski, Andrzej, of London.
Thompson, Crispin Felix, of Rangoon, Burma.
Venning, Cyril Geoffrey, of London.

Associate-Members

Andrzejewski, Tadeusz, of London.
Cassell, Joseph, of London.
Chow, Philip Yeong-Wai, of Singapore.
Dawson, Leonard Cecil, of Culcheth, nr. Warrington, Lancs.
Heath, Peter Geoffrey, of Birmingham.
Kulkarni, Shriniwas Rangnath, of Morvi, Saurashtra, India.
Maxwell, Michael, of Timperley, Cheshire.

MILLS, Gordon Manchester, B.Sc.(Tech.) Manchester, of Birmingham.
PLUNKET, Arthur Robert Lifford, of Sidcup, Kent.
RAMASWAMY, Bettadpur Venkata-Krishnappa, of Poona, India.
READ, Philip Charles, of London.
SREERAMULU, Anantharaju, of Madras, South India.
STAHL, Ludwig, A.M.I.C.E., of Johannesburg, South Africa.
TAUNTON, Arthur James, of London.
TAYLOR, Jack, of Coventry, Warwickshire.
WONG, Albert Peng Choon, of Singapore.
WOOD, Cyril Rudland, B.Sc.(Eng.) London, of London.

TRANSFERS

Students to Graduates

ALLCOCK, Arnold, of Salford, Lancs.
BOSWELL, Carlton, of Farnworth, Lancs.
CAMM, David Henry, of Pudsey, nr. Leeds.
HIGSON, Martin, of Manchester.
HILL, Peter Frederick, of Southend-on-Sea, Essex.
JARRETT, George Edward, of London.
JOHNSON, William Morris, of Harrogate, Yorks.
LACK, William Brian, of East Barnet, Herts.
MARTIN, Ronald Victor, of Loughton, Essex.
NEALE, David, of Swansea, Glam.
PERRY, Ronald Edward, of Germiston, Transvaal, South Africa.
POOLE, Liam Myles, of Durban, Natal, South Africa.
SYDENHAM, Richard Sidney, of Ilford, Essex.
THISTLETHWAITE, Gerard, of Ormskirk, Lancs.
THOMASON, William Brian, of Thames Ditton, Surrey.
TURKINGTON, William Kenneth Somme, of Belfast, Northern Ireland.
WATLING, Anthony John, of Barnet, Herts.
WHITWORTH, Brian, of Southport, Lancs.
WILKINSON, Herbert, of Grappenhall, nr. Warrington, Lancs.

Student to Associate-Member

BROWN, Robert Ross, of Harrogate, Yorks.

Graduates to Associate-Members

BACKHOUSE, Samuel Roy, A.M.I.C.E., of Glasgow.
BAILEY, Edwin Roy, of Merrow Common, nr. Guildford, Surrey.
BALSARA, Nariman Shapurji, B.E.(Civil), of Mbeya, Tanganyika Territory, B.E.A.
BARBER, Hubert, of Eccles, Lancs.
BARRETT, Guy Crosland, of Rawdon, nr, Leeds.
BUTTERS, Edward Albert, of London.
CHENG, HON KWAN, B.Sc., of Hong Kong.
CHESTER, Paul Stuart, A.M.I.Mun.E., of Malvern, Worcs.
DOYLE, Patrick Joyce, B.E.(N.U.I.), A.M.I.C.E., of Achiasi, Gold Coast.
DUROSE, George Arthur, of Liverpool, Lancs.
EVANS, Robert Edgar, of London.
FOAKES, Jack, of Cardiff.
FROST, Archibald Douglas, of London.
HAWORTH, Brian Singleton, B.Sc.(Eng.) Rand, of Salisbury, Southern Rhodesia.
HULFORD, Bertram William, of London.
JOSHIRAO, Chintaman Moreshwar, of London.
KELSALL, George Stuart, of Lytham St. Annes, Lancs.
LAKIN, Noel Oscar Ernest, B.E.(Civil) Calcutta, of Newcastle upon Tyne.
LANGDON, William Albert John, of Gloucester.
LEA, William Nigel, B.Sc.(Civil) Birmingham, of Norton Lindsey, Warwick.
MOHAMED, Shawky Hassan, of Cairo, Egypt.
MORAN, Thomas Francis, of Liverpool.
OWENS, Owen, B.Sc.(Eng.) Rand, of Livingstone, Northern Rhodesia.
PHILCOX, Keith Thomas, of Brighton.
PHILLIPS, John Harry, A.M.I.C.E., of Hemel Hempstead, Herts.
RIDEN, Alfred Donald, of Liverpool.
SINGH, Sandhu Pritam, B.Sc.(Eng.) Glasgow, of Portsmouth.
SRINIVASAN, Venkataramanan Kannurpatti, B.Sc., B.E., of Bombay, India.
STOCK, Walter Bernard, of Dewsbury, Yorks.
WEAVER, Charles Louis Harry, of London.
WILD, Peter, of Middlesbrough, Yorks.
WILLIAMS, Lionel Woolf, of Pinner, Middlesex.
WOOD, Frederick George, A.M.I.C.E., A.M.I.Mun.E., of Lytham St. Annes, Lancs.
WOTTON, Frederick Ernest, of London.

FORTHCOMING MEETINGS

The following meetings will be held at 11, Upper Belgrave Street, London, S.W.1.

Thursday, December 16th, 1954

Ordinary General Meeting for the election of members, at 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when Mr. M. F. Palmer, M.I.C.E., M.I.Struct.E., will give a paper on "Fabrication and Erection of Steel Plate Girder Railway Bridges."

Thursday, January 13th, 1955

Ordinary Meeting, 6 p.m., when Mr. W. R. Garrett, A.M.I.C.E., A.M.I.Struct.E. (Associate-Member of Council) will give a paper on "Gasholder Development and Design."

Thursday, January 27th, 1955

Ordinary General Meeting, 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when Mr. G. M. Boyd, A.M.I.Struct.E., M.I.N.A., will give a paper on "Brittle Fracture Problems in Steel Construction."

Thursday, February 10th, 1955

Ordinary Meeting, 6 p.m., when Mr. F. J. Samuely, B.Sc.(Eng.), M.I.Struct.E., A.M.I.C.E., will give a paper on "Structural Prestressing."

Thursday, February 24th, 1955

Joint Meeting with the Cement and Concrete Association and the Reinforced Concrete Association.

Ordinary General Meeting, 5.55 p.m., followed by a Joint Meeting with the Cement and Concrete Association and the Reinforced Concrete Association at 6 p.m., when Professor Mario Nervi will give a paper on "Some Reinforced Concrete Structures in Italy."

Members wishing to bring guests to the Ordinary and Joint Meetings referred to above, are requested to apply to the Secretary for tickets of admission.

EXAMINATIONS—JULY, 1954

The examinations were held in July, 1954, at the usual centres in Great Britain, and overseas at :—

Accra, Aden, Auckland, Baghdad, Beirut, Bloemfontein, Bombay, Bulawayo, Calcutta, Cape Town, Christchurch, Colombo, Cooma, Delhi (Aligarh), Dunedin, Durban, East London, Georgetown, B.G., Gold Coast (Achiasi), Hong Kong, Jerusalem, Johannesburg, Karachi, Kelantan, Kimberley, Kingston (Jamaica), Kisumu, Kuala Lumpur, Lagos, Lahore, Lusaka, Madras, Mbeya (Tanganyika), Melbourne, Mombasa, Montreal, Nairobi, Penang, Port Elizabeth, Rangoon,

Salisbury (Southern Rhodesia), Singapore, Sydney, Toronto, Trinidad, Wellington.

One hundred-and-ten candidates took the Graduateship Examination (79 at home, and 31 Overseas). Four hundred-and-eighty-three took the Associate-Membership Examination (371 at home, and 112 overseas). Of these, 67 passed the Graduateship Examination (49 at home, and 18 Overseas), and 84 passed the Associate-Membership Examination (73 at home, and 11 Overseas).

The names of the successful candidates are :—

GRADUATE EXAMINATION PASS LIST JULY, 1954

(Home Centres)

Allcock, Arnold, Bhar, Gurdarshan Singh, Boswell, Carlton, Camm, David Henry, Capps, Derrick Ernest, Cartlidge, Charles Peter, Cichonski, Feliks, crisp, Henry George, David, Evan Savours, Day, George William, Duczynski, Jerzy Leonard, Ejsymont, Zbigniew, Ettinger, Norman Barry, Farrow, Thomas Henry, Forest, Michael, Higson, Martin, Hill, Peter Frederick, Hillier, John Alan, Hollingum, Kenneth, Holt, Donald James, Husain, Mahmood, Jarrett, George Edward, Johnson, William Morris, Joseph, Anthony Errol, Kelsey, Peter John, Krauze, Franciszek, Kuczynski, Edward, Lack, William Brian, Mann, Edward Alexander, Martin, Ronald Victor, Mulholland, Bennett Douglas, Neale, David, Nurse, Colin Frederick, Pereira, Oscar Joseph, Perry, Mark, Roberts, John Merton, Sewell, Gordon Alfred Trevor, Singh, Jagdish, Stefanski, Andrzej, Sydenham, Richard Sidney, Thistlethwaite, Gerard, Thomason, William Brian, Turkington, William Kenneth Somme, Venning, Cyril Geoffrey, Wasiek, Alojzy Jan, Watling, Anthony John, White, Stanley Francis, Whitworth, Brian, Wilkinson, Herbert.

(Overseas Centres)

Baikoff, Eugene Marc Alexander, Day, Michael Chester, Gabay, Leslie Oliver, Ghaus, Mian Ghulan, Goh Kiew Yoh, Gould, Harold, Hanks, John Wilfred, Henderson, Leo Gordon, Higson-Smith, David John, Hughes, John William, Khoo Oon Lock, Maycroft, Douglas Harvey, Patrick, Colin Desmond, Perry, Ronald Edward, Poole, Liam Myles, Rodrigues, Philip Stanislaus, Thompson, Crispin Felix, Yagan, Joseph.

ASSOCIATE-MEMBERSHIP EXAMINATION PASS LIST—JULY, 1954

(Home Centres)

Andrzejewski, Tadeusz, Backhouse, Samuel Roy, Bailey, Edwin Roy, Barber, Hubert, Barrett, Guy Crosland, Bettany, George Angus, Bonnett, Clifford Frederick, Brown, Robert Ross, Butters, Edward Albert, Cassell, Joseph, Chester, Paul Stuart, Chow, Philip Yeong-Wai, Dawson, Leonard Cecil, Dickson, Hubert, Drage, John Frederick, Durose, George Arthur, Dziewulski, Jerzy, Evans, Robert Edgar, Fletcher, Kenneth Albert, Foakes, Jack, Frost, Archibald Douglas, Glover, Frank, Hawker, Geoffrey Fort, Heath, Peter Geoffrey, Heath, Roy Edward, Hewitt, Eric, Higson, Martin, Horn, Cyril Desmond, Hulford, Bertram William, Jablonski, Piotr, Joshirao, Chintaman Moreshwar, Kelsall, George Stuart, Kershaw, Geoffrey Philip, Knott, Stanley William, Kucieba, Jerzy Stefan, Lakin, Noel Oscar Ernest, Langdon, William Albert John, Lea, William Nigel, Luniewski, Zdzislaw, McCadden, Michael, Maxwell, Michael, Mills, Gordon Manchester, Mohamed, Shawky Hassan, Moran, Thomas Francis, Morton, Eric John, Philcox, Keith Thomas, Phillips, John Harry, Plunket, Arthur Robert Lifford, Prosser, William Henry, Read, Philip Charles, Riden, Alfred Donald, Singh, Sandhu Pritam, Stanley, Brian William, Steele, Alan David, Stock, Walter Bernard, Tate, Adrian Peter Kisse, Taunton, Arthur James, Taylor, Jack, Tobin, John Raymond, Walmsley, Joseph Roy, Weaver, Charles Louis Harry, Wedge, Henry Robert, Wild, Peter, Williams, Lionel Woolf, Winfield, Peter Frederick, Winiarski, Maciej Stanislaw, Wojciechowski, Jan, Wong, Albert Peng Choon, Wood, Cyril Rudland, Wood, Frederick George, Wotton, Frederick Ernest, Yeomans, Robert, Zelman, Maier I.

(Overseas Centres)

Balsara, Nariman Shapurji, Chan Weng Chiu, Cheng Hon Kwan, Doyle, Patrick Joyce, Haworth, Brian Singleton, Kulkarni, Shriniwas Rangnath, Owens, Owen, Ramaswamy, Bettadpur Venkata-Krishnappa, Sreeramulu, Anantharaju, Srinivasan, Venkataramanan Kannurpatti, Stahl, Ludwig.

EXAMINATIONS—JANUARY, 1955

The Examinations of the Institution will next be held at centres in the United Kingdom and overseas on January 11th and 12th, 1955 (Graduateship), and January 13th and 14th (Associate-Membership).

HONOURS AND AWARDS

In offering their sincere congratulations to the following members on the distinctions recently conferred upon them, the Council feel they are also expressing the good wishes of the Institution :—

Order of the British Empire—O.B.E.
Brevet-Major C. M. Spielman (Member).

Order of the British Empire—M.B.E.
Mr. F. T. West (Associate).

DRURY MEDAL AWARD

The fifth competition for the above award will take place in 1955. The subject is the design of a mobile crane.

Graduates and Students of the Institution who wish to compete are invited to apply for full details to the Secretary : envelopes to be marked in the top left-hand corner, " Drury Medal Award."

The closing date for the competition is October 1st, 1955.

The general conditions of the competition are as follows :—

1. The competition shall be for Graduates and Students of the Institution of not more than 25 years of age.

2. The subject of the competition shall be a design of a structural character, that is to say, primarily structural design, not planning.

3. The subject of design and conditions shall be prepared and issued biennially by a group of five members appointed by the Council.

4. The Literature Committee shall appoint a Jury of not less than five to examine the works submitted and to interview candidates, if found necessary.

5. In order to show that the work submitted is solely the work of the competitor, the documents submitted shall be countersigned by a corporate member of the Institution, or failing this, shall be accompanied by a declaration on a prescribed form signed by the candidate

in the presence of a Justice of the Peace or a Commissioner for Oaths.

LONDON GRADUATES' AND STUDENTS' SECTION

The following meetings will be held at 11, Upper Belgrave Street, London, S.W.1.

Wednesday, January 19th, 1955

Annual Dance.

Tuesday, January 15th, 1955

Address by the President of the Institution.

BRANCH NOTICES

LANCASHIRE AND CHESHIRE BRANCH

The following meetings have been arranged :—

Tuesday, December 7th, 1954

Trip to John Summers & Sons' Steelworks, Shotton, by the Liverpool members of the Branch.

Friday, January 7th, 1955

At the College of Technology, Manchester, 6.30 p.m. Dr. P. W. Rowe, B.Sc., A.M.I.C.E. (Graduate), on "The Present Situation on Retaining Wall Design."

Tuesday, January 11th, 1955

A Joint Meeting with the Institute of Welding (Liverpool and District Branch) will be held at the Liverpool City College of Technology, Liverpool, at 7 p.m., when Mr. L. Rotherham will give a paper on "Welding at the Atomic Plants."

Tuesday, January 25th, 1955

At the College of Technology, Manchester, 6.30 p.m. Dr. T. Howarth, A.R.I.B.A., on "Modern Architecture."

Tuesday, February 1st, 1955

Joint Meeting with the Reinforced Concrete Association, at the Liverpool Engineering Society, Dale Street, Liverpool, at 6.30 p.m. Dr. D. D. Matthews, M.A., D.Eng., M.Sc.(Eng.), M.I.Struct.E., A.M.Am.Soc.C.E. (Member of Council) will give a paper on "Prestressed Concrete Bridge at Nottingham."

Monday, February 14th, 1955

Joint Meeting with the Institution of Civil Engineers, at the College of Technology, Manchester, at 6.30 p.m., when Mr. T. C. Waters, M.I.Struct.E. will give a paper on "The Process Building at Capenhurst."

Monday, February 28th, 1955

At the College of Technology, Manchester, at 6.30 p.m., Mr. D. R. R. Dick, B.Sc., M.I.C.E. on "The Design and Construction of the Nuclear Reactor Buildings at Windscale Works, Sellafield."

Joint Hon. Secretaries : A. S. Sinclair, A.M.I.Struct.E., 17, The Circuit, Cheadle Hulme, Cheshire ; M. D. Woods, A.M.I.Struct.E., 58, Spring Gardens, Salford 6, Lancs.

MIDLAND COUNTIES BRANCH

The following meetings have been arranged :—

Tuesday, January 4th, 1955

Joint Meeting with the Reinforced Concrete Association, Midland Counties Branch, at the Midland Institute, Birmingham, 6 p.m., Dr. F. G. Thomas, Ph.D., B.Sc., M.I.C.E., M.I.Struct.E. (Member of Council), on "Load Factor Methods of Design of Reinforced Concrete."

Friday, January 28th, 1955

Joint Meeting with the Institute of Welding, Midland Section, at the James Watt Memorial Institute, Great Charles Street, Birmingham, at 6 p.m., Mr. S. M. Reisser, B.Sc., M.I.Struct.E., A.M.I.C.E., will give a paper on "The Influence of Welding on Steel Building Structures with Particular Reference to Erection."

Tuesday, February 8th, 1955

At the Supper Room, The King's Hall, Queen Street, Derby, at 7 p.m., Mr. F. Brooksbank, M.A.(Cantab.) (Graduate), on "Economics in Welding Design."

Friday, February 25th, 1955

At the Department of Civil Engineering, The University, Birmingham, at 6 p.m., Professor S.C. Redshaw, Ph.D., D.Sc., M.I.C.E., F.R.Ae.S., on "Modern Methods of Experimental Stress Analysis in Structural Engineering."

Hon. Secretary : L. A. Firminger, A.M.I.Struct.E., 656, Chester Road, Erdington, Birmingham, 23.

MIDLAND COUNTIES GRADUATES' AND STUDENTS' SECTION

The following meetings have been arranged :—

Monday, January 31st, 1955

Joint Meeting with the Junior Section of the Birmingham and Five Counties Architectural Association, at the James Watt Memorial Institute, Great Charles Street, Birmingham, at 6 p.m., Mr. H. V. Hill, M.Sc., A.M.I.C.E., A.M.I.Struct.E. (Associate-Member of Council), will give a paper on "The Use of Light Alloys in Structures."

Hon. Secretary : A. K. A. Costain, A.M.I.C.E., 134, Witherford Way, Weoley Hill, Birmingham, 29.

NORTHERN COUNTIES BRANCH

The following meetings have been arranged :—

Wednesday, December 1st, 1954

At the Neville Hall, Westgate Road, Newcastle upon Tyne, at 6.30 p.m., Mr. A. P. Clarke, B.Sc., will give a paper on "Lackenby Steelworks."

Tuesday, December 7th, 1954

At the Cleveland Scientific and Technical Institution, Corporation Road, Middlesbrough, at 6.30 p.m., Mr. E. Czeiler, M.I.Struct.E., will give a paper on "Steelwork for Hindhaugh Street Flats, Newcastle."

Wednesday, January 12th, 1955

Joint Meeting with the Institution of Civil Engineers, at the Cleveland Scientific and Technical Institution, Corporation Road, Middlesbrough, at 6.30 p.m., when Dr. A. W. Skempton, D.Sc., A.M.I.C.E., will give a paper on "Some Practical Applications of Soil Mechanics."

Thursday, January 13th, 1955

Joint Meeting with the Northern Architectural Association, at Higham Place, Newcastle, at 7.30 p.m.

Tuesday, February 1st, 1955

At the Cleveland Scientific and Technical Institution, Corporation Road, Middlesbrough, at 6.30 p.m., Mr. F. A. Partridge, B.Sc., A.C.G.I., M.I.C.E., on "The Plastic Theory."

Wednesday, February 2nd, 1955

The above meeting will be repeated at the Neville Hall, Westgate Road, Newcastle upon Tyne, at 6.30 p.m.

Hon. Secretary : Captain O. Lithgow, A.M.I.Struct.E., 4, Stoneleigh Avenue, Acklam, Middlesbrough.

NORTHERN IRELAND BRANCH

The following meetings have been arranged :—

Tuesday, December 7th, 1954

At the College of Technology, Belfast, at 6.45 p.m., Mr. W. S. Atkins, B.Sc., M.I.C.E., M.Inst.W., will give a paper on " Structural Steelwork and Concrete Construction, with Particular Reference to Abbey Steelworks."

Tuesday, January 4th, 1955

At the College of Technology, Belfast, at 6.45 p.m., Mr. J. D. Boyd, B.Sc., will give a paper on " Reinforced Concrete Portal Frames in Northern Ireland."

Tuesday, February 1st, 1955

At the College of Technology, Belfast, at 6.45 p.m., Mr. D. V. Pike, M.I.Struct.E., A.M.I.C.E., will give a paper on " Structural Uses of Aluminium."

Hon. Secretary : A. H. K. Roberts, B.A., B.A.I., M.I.Struct.E., M.I.C.E.I., " Barbizon," 26, Dunlambert Park, Belfast.

SCOTTISH BRANCH

The following meetings have been arranged :—

Friday, December 3rd, 1954

Joint Meeting with the East of Scotland Branches of The Institute of Welding and the Society of Engineers, at the Heriot Watt College, Chambers Street, Edinburgh. Paper on " Welded Structures."

Tuesday, January 18th, 1955

At the Institution of Engineers and Shipbuilders, 39, Elmbank Crescent, Glasgow, at 7 p.m., Mr. Hugh Fraser, B.Sc., M.I.Struct.E., A.M.I.C.E., and Mr. W. G. Cantlay, B.Sc., A.M.I.C.E., A.M.I.Struct.E., will give a paper on " A Graphical Approach to the Design of Two and Three Span Rigid Frame Buildings."

Friday, February 11th, 1955

Joint Meeting with the Glasgow and West of Scotland Association of The Institution of Civil Engineers, at The Institution of Engineers and Shipbuilders, 39, Elmbank Crescent, Glasgow, at 7.15 p.m., when Mr. F. A. Partridge, B.Sc.(Eng.), A.C.G.I., A.M.I.C.E., will give a paper on " The Plastic Theory as Applied to the Design of Mild Steel Structures."

Hon. Secretary : G. Drysdale, A.M.I.Struct.E., " Niaroo," 33, Union Street, Motherwell, Lanarkshire.

SOUTH-WESTERN COUNTIES BRANCH

Friday, March 18th, 1955

At Exeter, at 7 p.m., Mr. F. E. Somerset, on " The Efficiency of the Tube Section." (Accompanied by a film.)

Joint Hon. Secretaries: E. W. Howells, M.I.Struct.E., 10-12, Market Street, Torquay ; C. J. Woodrow, " Elstow," Hartley Park Villas, Tavistock Road, Plymouth, Devon.

WALES AND MONMOUTHSHIRE BRANCH

The following meetings have been arranged :—

Wednesday, December 8th, 1954

At the Mackworth Hotel, Swansea, at 6.30 p.m., Mr. H. E. Lewis, B.Sc., D.I.C. (Graduate), will give a paper on " Developments in the Structural Use of Concrete."

Monday, January 31st. 1955

Joint Meeting with the Institution of Civil Engineers, at the South Wales Institute of Engineers, Park Place, Cardiff, at 6.30 p.m., when Dr. B. G. Neal, A.M.I.C.E., will give a paper on " Simple Plastic Theory."

Wednesday, February 16th, 1955

Junior Members' Evening at the Mackworth Hotel, Swansea, at 6.30 p.m.

Hon. Secretary : K. J. Stewart, A.M.I.C.E., A.M.I.Struct.E., 15, Glanmor Road, Swansea.

WESTERN COUNTIES BRANCH

The following meetings have been arranged :—

Friday, December 3rd, 1954

At the University of Bristol Geology Lecture Theatre (entrance University Road), at 6 p.m., Mr. J. Guthrie Brown, M.I.C.E., M.I.Struct.E. (Vice-President), will give a paper on " Highlights in an Engineer's Life."

Friday, December 10th, 1954

Combined Dance, Royal Hotel, Bristol.

Friday, January 7th, 1955

At the University of Bristol Geology Lecture Theatre, at 6 p.m., Mr. H. G. Lakeman, A.C.G.I., B.Sc., Eng. (London), A.M.I.C.E., will give a paper on " Recent Bridge Works in Bristol Dictrict."

Friday, February 4th, 1955

At the University of Bristol Geology Lexture Theatre, at 6 p.m., Mr. Clifford E. Saunders, M.I.Struct.E., will give a paper on " Some Effect of Prefabrication on Post-War Buildings."

Wednesday, February 16th, 1955

Annual Dinner at the Royal Hotel, Bristol.

Hon. Secretary : E. Hughes, A.M.I.Struct.E., 23, Southdown Road, Westbury-on-Trym, Bristol, 9.

YORKSHIRE BRANCH

The following meetings have been arranged :—

Wednesday, December 15th, 1954

At the Great Northern Hotel, Leeds, at 6.30 p.m., Mr. A. P. Clark and Mr. T. V. Thompson, M.I.Struct.E., will give a paper on " Lackenby Steelworks."

Wednesday, January 19th, 1955

At the Great Northern Hotel, Leeds, at 6.30 p.m., Mr. F. A. Charman, B.Sc., A.M.I.C.E., will give a paper on " The Construction of The Woodhead New Tunnel."

Wednesday, February 16th, 1955

At the Great Northern Hotel, Leeds, at 6.30 p.m., Mr. F. N. Sparkes, M.Sc., M.I.C.E., will give a paper on " The Principles of Quality Control."

Hon. Secretary : E. Wrigley, A.M.I.Struct.E., 17, The Drive, Alwoodley, Leeds.

UNION OF SOUTH AFRICA BRANCH

Hon. Secretary : A. E. Tait, B.Sc., A.M.I.C.E., P.O. Box 3306, Johannesburg, South Africa.

During week-days Mr. Tait can be contacted in the City Engineer's Department, Town Hall, Johannesburg. Phone : 34-1111, Ext. 257.

Natal Hon. Secretary : E. G. Bennett, A.M.I.Struct.E., c/o The Reinforcing Steel Co., Ltd., P.O. Box 49, Merebank, Durban.

Cape Section Hon. Secretary : R. Stubbs, M.I.Struct.E., The Reinforcing Steel Co., Ltd., P.O. Box 2962, Cape Town.